GEORGIA INSTITUTE OF TECHNOLOGY
OFFICE OF CONTRACT ADMINISTRATION
SPONSORED PROJECT INITIATION

18

Date: 10/4/77

Project Title: "Strengthening of Existing Bridges Using Epoxy Injection."

Project No: E-20-625

Project Director: Dr. L. F. Kahn and Dr. K. M. Will

Sponsor: Georgia Department of Transportation

Agreement Period: From 9/13/77 Until 8/12/78

Type Agreement: Contract (GDOT Project No. 7702)

Amount: $70,093

Reports Required: Quarterly Progress Reports, Interim Reports, Final Report.

Sponsor Contact Person (s):

Technical Matters

Contractual Matters
(thru OCA)

Mr. Rick Deaver or
Mr. Paul Liles
GEORGIA DEPARTMENT OF TRANSPORTATION
Office of Materials and Research
Research and Development Bureau
15 Kennedy Drive
Forest Park, GA 30050
(404) 363-7583

Defense Priority Rating: N/A

Assigned to: Civil Engineering (School/Laboratory)

COPIES TO:

- Project Director
- Division Chief (EES)
- School/Laboratory Director
- Dean/Director—EES
- Accounting Office
- Procurement Office
- Security Coordinator (OCA)
- Reports Coordinator (OCA)
- Library, Technical Reports Section
- Office of Computing Services
- Director, Physical Plant
- EES Information Office
- Project File (OCA)
- Project Code (GTRI)
- Other __________

CA-3 (3/76)

GEORGIA INSTITUTE OF TECHNOLOGY
OFFICE OF CONTRACT ADMINISTRATION

SPONSORED PROJECT TERMINATION

Date: 1/8/81

Project Title: Strengthening of Existing Bridges Using Epoxy Injection

Project No: E-20-625

Project Director: Dr. Will/Dr. Kahn

Sponsor: Georgia Department of Transportation

Effective Termination Date: 4/30/79

Clearance of Accounting Charges: 4/30/79

Grant/Contract Closeout Actions Remaining:

NONE

- _ Final Invoice and Closing Documents
- _ Final Fiscal Report
- _ Final Report of Inventions
- _ Govt. Property Inventory & Related Certificate
- _ Classified Material Certificate
- _ Other ______________________

Assigned to: Civil Engineering (School/~~Laboratory~~)

COPIES TO:

Project Director
Division Chief (EES)
School/Laboratory Director
Dean/Director—EES
Accounting Office
Procurement Office
Security Coordinator (OCA)
Reports Coordinator (OCA)
Library, Technical Reports Section
EES Information Office
Project File (OCA)
Project Code (GTRI)
Other Project Code (OCA)
OCA Research Property Coordinator

E-20-625

RESEARCH QUARTERLY PROGRESS REPORT

GEORGIA DEPARTMENT OF TRANSPORTATION

Date of Report: January 10, 1978

1 Project No. State/Agency	2 Project Title	3 Quarterly Report
7702	Strengthening of Existing Bridges Using Epoxy Injection	No. 1 From Sept 13, 1977 To Dec 31, 1977

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology Atlanta, Georgia	Lawrence F. Kahn, Assistant Prof. Kenneth M. Will, Assistant Prof.

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept 13, 1977	Aug 12, 1978	33	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended				
11 Total	12 Current Fiscal Year	13 Total to Date	%	14 Current Fiscal Year	%	15 Report Quarter
$70,093	$70,093	$6866.70	9.8	$6866.70	9.8	$6866.70

16 Project Schedule — Time Period

Research Tasks	1977: S O N D — 1978: J F M A M J J A S O	% Task Completed
Analytical Studies		100
Development of Slab Jacking Apparatus		100
Jacking of Slab & Epoxy Injection		100
Removal of Slab by GDOT		0
Interim Report Phase I		0
Phase II		0
Phase III		0
Final Report		0
Overall % Completed	10	

☐ Approved Schedule   Work Completed Schedule   Projected Completion Schedule

17 Progress This Quarter (By Task)

Analytical Studies - The analytical studies have resulted in a table which lists the increase in moment of inertia and section modulus for original noncomposite steel beam/concrete deck sections which are made composite. A computer program has been written to yield the section properties.

Slab Jacking Apparatus - An apparatus was developed to jack the slab from the supporting girders. On one girder, the slab was lifted off with 1/16 and 1/32 inch steel shims, placed between the slab and girder. On another girder, the slab was lifted off and set back on the girder while on a third girder the slab was not removed.

Epoxy Injection - Copper tube ports for the epoxy injection were installed. Around the ports and along the flange-slab interface, cracks were sealed with epoxy. Low viscosity epoxy was pressure injected between the slab and girder. The stripping of the slab has been delayed so that the injected epoxy has time to cure.

18 Work Planned for Next Quarter

Removal and inspection of slab segment injected in Phase I. Submittal of interim report. Subject to results observed when slab removed and availability of suitable bridges, Phase II activities will begin.

19 Significant Technical Information, Recommendations, Implementation

Experience obtained during Phase I activities indicates that epoxy injection should not be performed during periods of cold temperature. The cold temperature inhibits epoxy bonding.

20 Problems

Delay in completion of Phase I activities are due to (1) problems encountered when jacking slab, (2) cold weather conditions, and (3) scheduling epoxy subcontractor.

21 Report Prepared by

Signature	Name	
[signature]	Kenneth M. Will	Assistant Professor, Co-Project Director

E 20-625

RESEARCH QUARTERLY PROGRESS REPORT

GEORGIA DEPARTMENT OF TRANSPORTATION

Date of Report

April 10, 1978

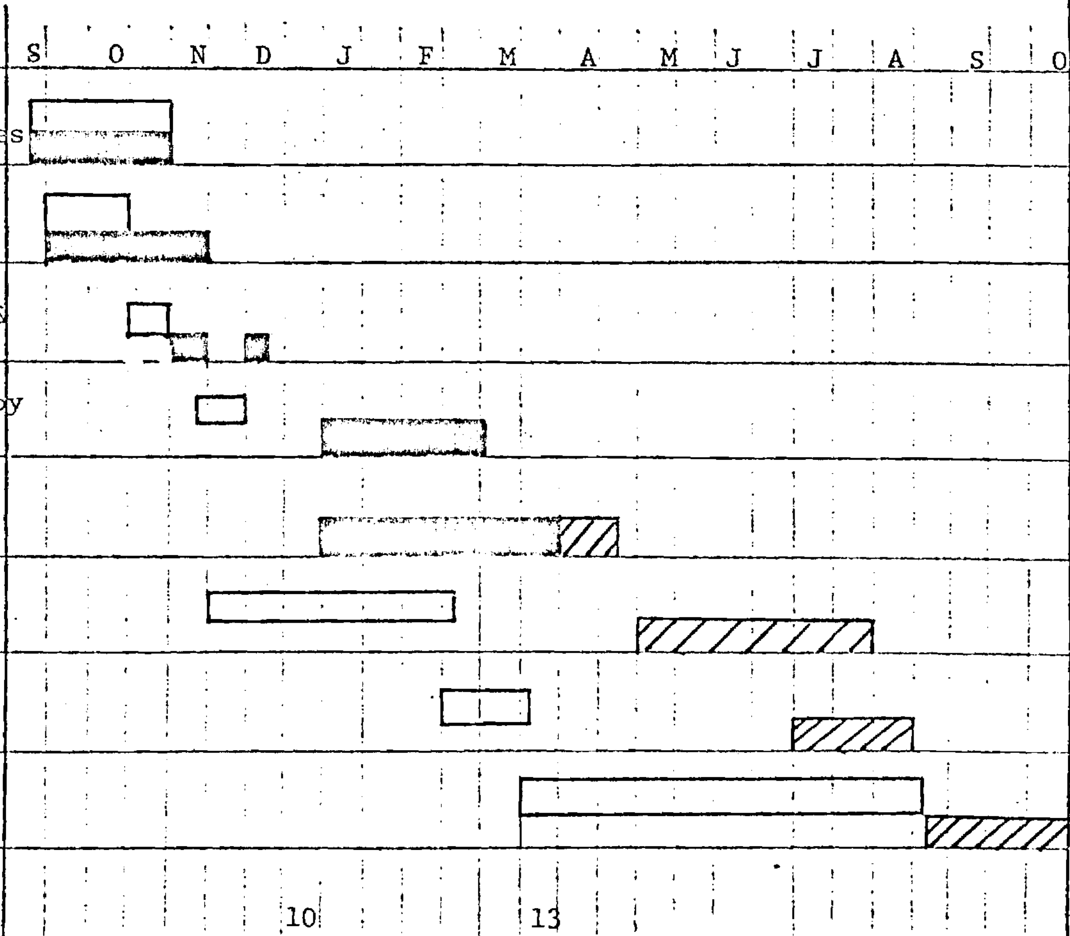
1 Project No. State/Agency	2 Project Title	3 Quarterly Report No. 2
7702	Strengthening of Existing Bridges Using Epoxy Injection	From January 1, 1978 To March 31, 1978

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology Atlanta, Georgia	Lawrence F. Kahn, Asst. Prof. Kenneth M. Will, Asst. Prof.

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 13, 1977	Aug. 12, 1978	60	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended				
11 Total	12 Current Fiscal Year	13 Total to Date	%	14 Current Fiscal Year	%	15 Report Quarter
$70,093	$70,093	$8,724	13	$8,724	13	$2,072

16 Project Schedule — Time Period



Research Tasks	% Task Completed
Analytical Studies	100
Development of Slab Jacking ~~Apparatus~~	100
Jacking of Slab & Epoxy Injection	100
Removal of Slab by GDOT and GT	100
Interim Report Phase I	50
Phase II	0
Phase III	0
Final Report	0
Overall % Completed	

17 Progress This Quarter (By Task)

The primary activity of this quarter was the removal of the slab sections on the Camp Creek Parkway bridge where epoxy was injected during previous quarter. The slab was sawed in sections over the girders where injection was performed by GDOT. Three methods of removing the slab sections were used. The first method developed by GDOT was unsuccessful due to eccentricity of the loading which did not produce a clean separation. The second was the method used for lifting the slab prior to injection. This method was moderately successful although it introduced bending into the slab section. The third method utilized jacks in the same plane as the slab. This produced primarily shear stresses on the slab-girder interface. Clean separation of the slab segments resulted using this method. Although the work plan indicated that only visual inspections were to be performed, the third method provided a quantitative measure of the shear stress on the slab-girder interface.

18 Work Planned for Next Quarter

Interim report for Phase I activities and a modified work plan for Phases II and III will be submitted. Depending on acceptance of modified work plan, Phase II activities will begin.

19 Significant Technical Information, Recommendations, Implementation

The removal of the slab sections indicated that there was considerable bond due to the natural adhesion between the concrete and steel. The magnitude of this bond is of the same order as the epoxy bond in those areas where injection was successful.

20 Problems

Delays in the cutting of the slab due to cold weather in January.

21 Report Prepared by

Kenneth M. Will

Co-project Director
Assistant Professor

E-20-625

RESEARCH QUARTERLY PROGRESS REPORT GEORGIA DEPARTMENT OF TRANSPORTATION	Date of Report 17 July 1978

1 Project No. State/Agency	2 Project Title	3 Quarterly Report No. 3
7702	Strengthening of Existing Bridges Using Epoxy Injection	From April 1, 1978 To June 30, 1978

4 Research Agency	5 Project Director(s)
Georgia Institute of Technology Atlanta, Georgia	Lawrence F. Kahn, Asst. Prof. Kenneth M. Will, Asst. Prof.

6 Starting Date	7 Completion Date	8 % Time Expended	9 Schedule Status	10 Sufficiency of Funds
Sept. 13, 1977	Aug. 12, 1978	87	☐ Ahead ☒ Behind ☐ On	☒ Sufficient ☐ Insufficient

Funds Authorized		Funds Expended				
11 Total	12 Current Fiscal Year	13 Total to Date	%	14 Current Fiscal Year	%	15 Report Quarter
$70,093	$70,093	9569	14	9569	14	132

16 Project Schedule

Research Tasks	Time Period (S O N D J F M A M J J A S O)	% Task Completed
Analytical Studies		100
Development of Slab Jacking Apparatus		100
Jacking of Slab & Epoxy Injection		100
Removal of Slab by GDOT and GT		100
Interim Report Phase I		100
Phase II *		0
Phase III *		0
Final Report *		0
Overall %	* Completion depends on modified work plan acceptance date. 10 13 14	

Approved Schedule   Work Completed Schedule   Projected Completion Schedule

E-20-625

STRENGTHENING OF EXISTING BRIDGES USING EPOXY INJECTION

PHASE I INTERIM REPORT

GEORGIA DEPARTMENT OF TRANSPORTATION
PROJECT NO. 7702

by

L. F. Kahn, K. M. Will, P. H. Sanders, and J. A. Soltesz

School of Civil Engineeering

Georgia Institute of Technology

Atlanta, Ga. 30332

June, 1978

STRENGTHENING OF EXISTING BRIDGES USING EPOXY INJECTION

PHASE I INTERIM REPORT

by

L. F. Kahn, K. M. Will, P. H. Sanders, and J. A. Soltesz

Prepared for Georgia Department of Transportation Project No. 7702

ACKNOWLEDGMENTS

This research has been performed in connection with the project "Strengthening of Existing Highway Bridges Using Epoxy Injection", sponsored by the Georgia Department of Transportation Project No. 7702.

Mr. Hugh L. Tyner, Chief of Research and Development provided technical advise in several stages at this project. State Bridge Engineer Mr. J. T. Kratzer performed studies relating the number of bridges for which epoxy injection was applicable.

Mr. Rick Deaver of the Georgia's DOT Research group assisted in suppling background information and contributed in many phases of the field investigations.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

STRENGTHENING OF EXISTING BRIDGES USING EPOXY INJECTION
PHASE I INTERIM REPORT

1.0 INTRODUCTION

1.1 Purpose

The overall objective of the three-phase research project is to determine if existing non-composite highway bridges can be made composite by injecting epoxy between the steel stringers and concrete deck. The specific purpose of the first phase reported herein was to establish the feasibility of strengthening existing bridges using epoxy injection. Both analytic and experimental investigations were used to judge this feasibility. The experimental study was concerned with the general applicability and technique of the epoxy grouting method and with the apparent strength of the resulting girder-to-deck bond.

1.2 Scope

The scope of Phase I was limited. The analytic study investigated the increase in section modulus of non-composite girder-slab structures if they were made composite, the anticipated strength increase for three I-85 bridges being considered for the experimental study, and the increase in service load for four existing non-composite spans which were designed for H-15 loadings.

A single bridge over I-85 near Hartsfield International Airport, south of Atlanta, Georgia, was selected for the experimental investigation because it was abandoned and was scheduled for demolition. Three different methods for epoxy injection were tried over 10-ft. lengths of three interior stringers of one span of the bridge. After epoxy injection, the slabs over the stringers were removed. Results obtained from the removal of the deck showed both quantitatively and qualitatively the degree of adhesion of the girder to the slab.

1.3 Background

Epoxy injection previously has not been used to bond existing steel girders to concrete slabs. Some research has shown that composite beams may be made by epoxy bonding steel girders or plates to concrete segments as reported in detail in Appendix A. This previous research indicated that epoxy injection had the potential for bonding existing girders and slabs.

Appendix A discusses the need for strengthening of existing bridges throughout the nation. Research by Mr. J. T. Kratzer and Mr. F. A. Childers, State Bridge and Maintenance Engineers, respectively, for the Georgia Department of Transportation, indicates that within Georgia there are 898 simple steel span bridges that have design loads of H-10 or H-15. Approximately 1082 bridges are simple-span non-composite steel girders with concrete decks. These facts indicate that hundreds of bridges within Georgia may be candidates for strengthening by providing complete or partial composite action between the steel stringers and concrete deck.

2.0 EXPERIMENTAL INVESTIGATION

2.1 Bridge Selection

Three bridges were made available for experimental research by the abandonment of a section of I-85 near Hartsfield Airport, south of Atlanta, Georgia. The Camp Creek Parkway bridge was selected for the Phase I research for several reasons. The bridge was away from immediate airport expansion, and therefore would not be demolished for many months; it was simple span which would require simpler and less expensive instrumentation should the same structure be used for load tests; it was a non-composite structure.

Figure 1 shows the 117-ft. span of the Camp Creek Parkway bridge used for the investigation, while Figure 2 illustrates the cross-section. The nominal 5-ft. deep girders had flanges 18 in. wide with thicknesses which varied between 1-3/8 to 1-7/8 in. As shown the three interior girders were selected so that the deck lifting apparatus could be fabricated to span between the bottom flanges of the stringers.

Analysis indicated that if the non-composite section of the bridge were made fully composite the section modulus would be increased by 10 percent. While such a small increase would make data interpretation from load tests difficult, it would not influence the evaluation of epoxy injection techniques which was the principal objective of Phase I.

2.2 Girder-Slab Separation

One of the most important variables of the epoxy injection procedure was the space between the steel girder and the concrete slab into which the epoxy would be injected. From the authors' previous experience with epoxy bonding, it was estimated that a minimum gap needed for injection was about 1/100 in. and that 1/32 in. was desirable. To raise the slab and to shim it off the girders was considered to be the least advantageous method for developing a gap because in a full-scale injection procedure such deck raising would be an expensive process. The most advantageous situation would be if a "natural" separation existed which would permit injection without a preliminary separation process.

The separation variable was investigated by using three different separation techniques for Girders 1, 2, and 3 respectively. For Girder 1 the slab was not raised from the girder; the attempt would be made to inject epoxy into any

Figure 1: Camp Creek Parkway Bridge

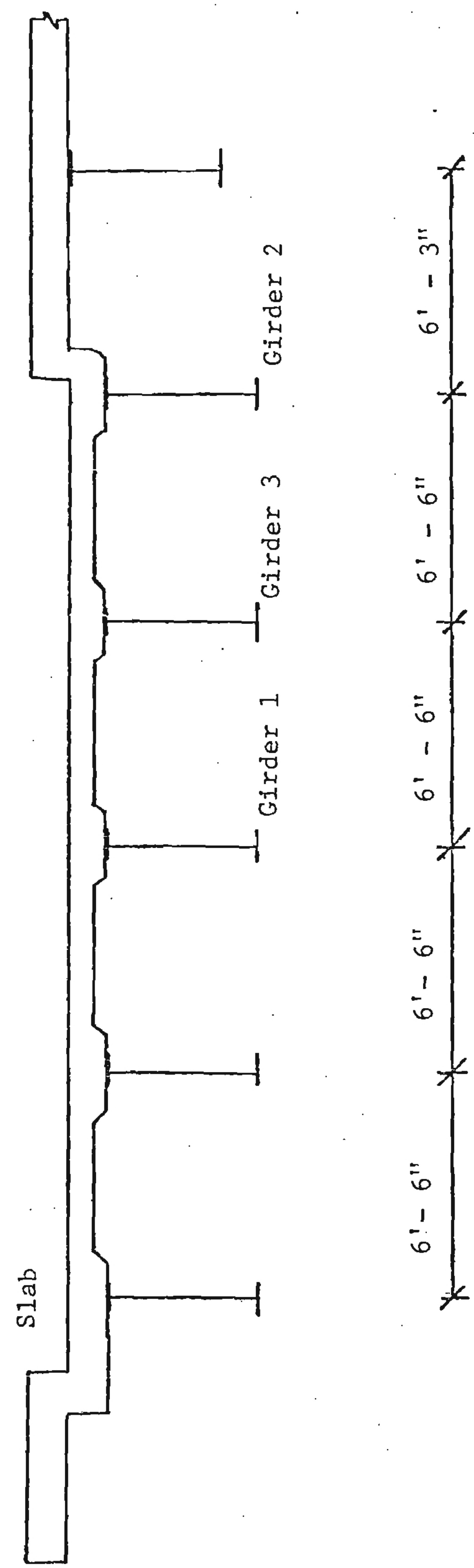


Figure 2: Cross Section of Camp Creek Parkway - 117' span.

"natural" separation which existed. For Girder 2 a separation was created by raising the slab off the girder and then by lowering the slab back onto the girder. The gap was judged to be small, but it was assured that any chemical bond between the concrete and steel was broken. For Girder 3 the slab was raised and shimmed so that over a center portion of the test area the gap was 1/16 in. and over the end portions the gap was 1/32 in.

2.2.1 Jacking Apparatus

In order to raise the slab off Girders 2 and 3, a hydraulic jacking system was developed. Details of the jacking system are presented in Appendix B. Figure 3 illustrates the jacking apparatus. Two W10 or two W12 steel beams were used to span between Girders 1 and 3, and between Girders 3 and 2. Hydraulic jacks were mounted on bearing plates which were bolted to the beams. Pipe columns extended from the jacks to the underside of the deck. Bearing plates against the deck were joined to the columns by spherical bearings which assured concentric loading of the columns and which allowed for variations in the slab surface.

2.2.2 Jacking Procedure

The slab over Girder 3 was raised first by pressurizing Jacks A and B as shown in Figure 3. At a load of 32 kips in Jack A and of 30 kips in Jack B, the slab began to rise; the load decreased even though the slab was raised farther. The maximum separation was 1/4 in. adjacent to the jack location; the separation decreased away from the jacks with no gap at a distance of about 5 ft.

Coping on the underside of the slab cracked and fell from the slab as it was raised off Girder 3. Friction between the edge of the flange and coping caused the fracture. With the coping removed, the separation was clearly visible and accessible.

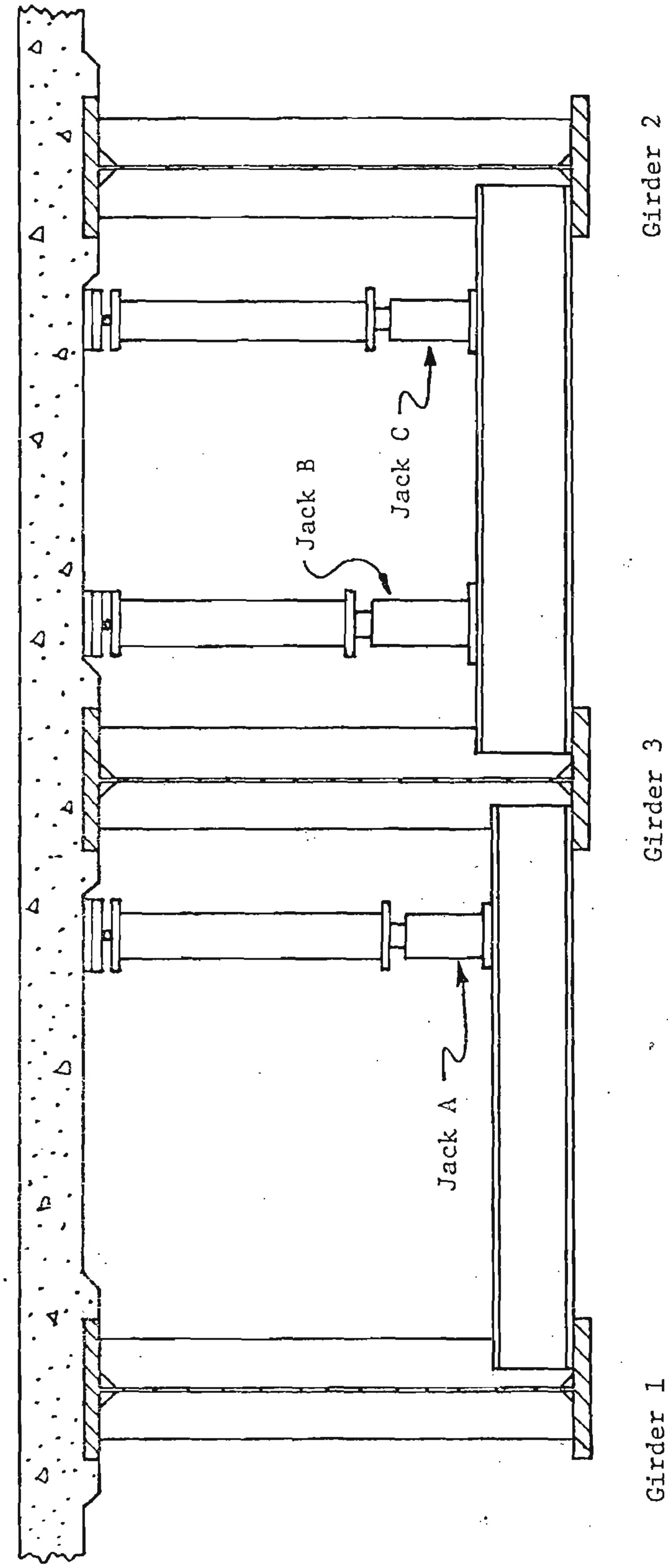


Figure 3. Loading apparatus, cross-section of bridge looking east.

Shims were placed in the separation over Girder 3. The jacking apparatus was located in two positions along the length of the girders so that the slab could be raised for a total length of 12 ft. Shims 1/32 in. thick were placed at 16 in. spacing near the ends of this length, and 1/16 in. shims were placed at the center. The shims extended 17 in. across the 18 in. wide flange. The slab was lowered when the shims were in place.

The slab over Girder 2 was raised by pressurizing jacks B and C which were located on the north side of the girder. After the slab was raised a maximum of 1/4 in., it was returned to the girder; no shims were placed. The jacking apparatus was located in two positions along the length of the girders so that the slab was raised for a total length of 10 ft. over Girder 2.

Coping along Girder 2 did not crack and fall; therefore, the separation was not visible.

The slab over Girder 1 was not raised. Jacking of the slab over the other girders did not seem to influence the slab and coping over Girder 1.

2.3 Epoxy Injection Procedure

The epoxy injection procedure involved attaching injection ports to the area to be bonded, sealing all cracks, and injecting a low viscosity epoxy through the ports.

2.3.1 Port Installation

The ports were made of copper tubing with a nominal 1/4 in. outside diameter so that a standard plumbing pressure fitting could be used for attachment to the epoxy injection equipment. The epoxy injection ports were positioned so that the epoxy flowed into the gap between the bottom of the slab and the top of the flange of the girder. Where the coping remained next to the flange (Girders 1 and 2), a 1/4 in. diameter hole was drilled vertically into the concrete.

The hole was adjacent to the flange; the edge of the flange served as a guide. For the two girders with no shims, ports were installed on both sides of the flange at 2'-0" spacing; their positions were alternated on opposite sides of the girder so that ports were at 1-ft. intervals along the length of the girder.

The copper tubes were cut 6 in. long, and on one end a lengthwise slice was made. This slice facilitated flow of the epoxy out of the tube and into the girder-slab joint. The tubes were bonded in place using an epoxy of putty-like consistency (Sika Dur-Gel, a two-part 100 percent solids epoxy). Figures 4 and 5 show the finished ports for Girder 1 and 2. Application of the sealer around the ports is shown in Figure 6.

On Girder 3 the coping on one side had been removed; therefore, the gap was clearly exposed and no holes were drilled on that side. The sliced end of the copper tube was pressed against the gap as shown in Figure 7, and the gel epoxy was used to bond it in place. One tube was placed between each set of shims, approximately at 16-in. spacing. A single hole was drilled into the coping on the other side of the shimmed girder, and a copper tube inserted. This tube was later used to inspect whether the injected epoxy flowed from one side of the flange to the other side. A diagram of the shimmed girder ports is presented in Figure 8. Air was blown into the ports to remove dust.

2.3.2 Sealer Application

After placement of the tubes, all cracks and gaps on each side of the girders were sealed with the gel epoxy to prevent injected epoxy from leaking.

Damp and cold weather conditions inhibited the sealing procedure. The steel was dried using a towel and electric 1000 watt dryer. Sealer was applied all along the area where the concrete and steel coping came into contact with the top flange (Figures 9 and 10), and to any other area where epoxy might be expected to leak.

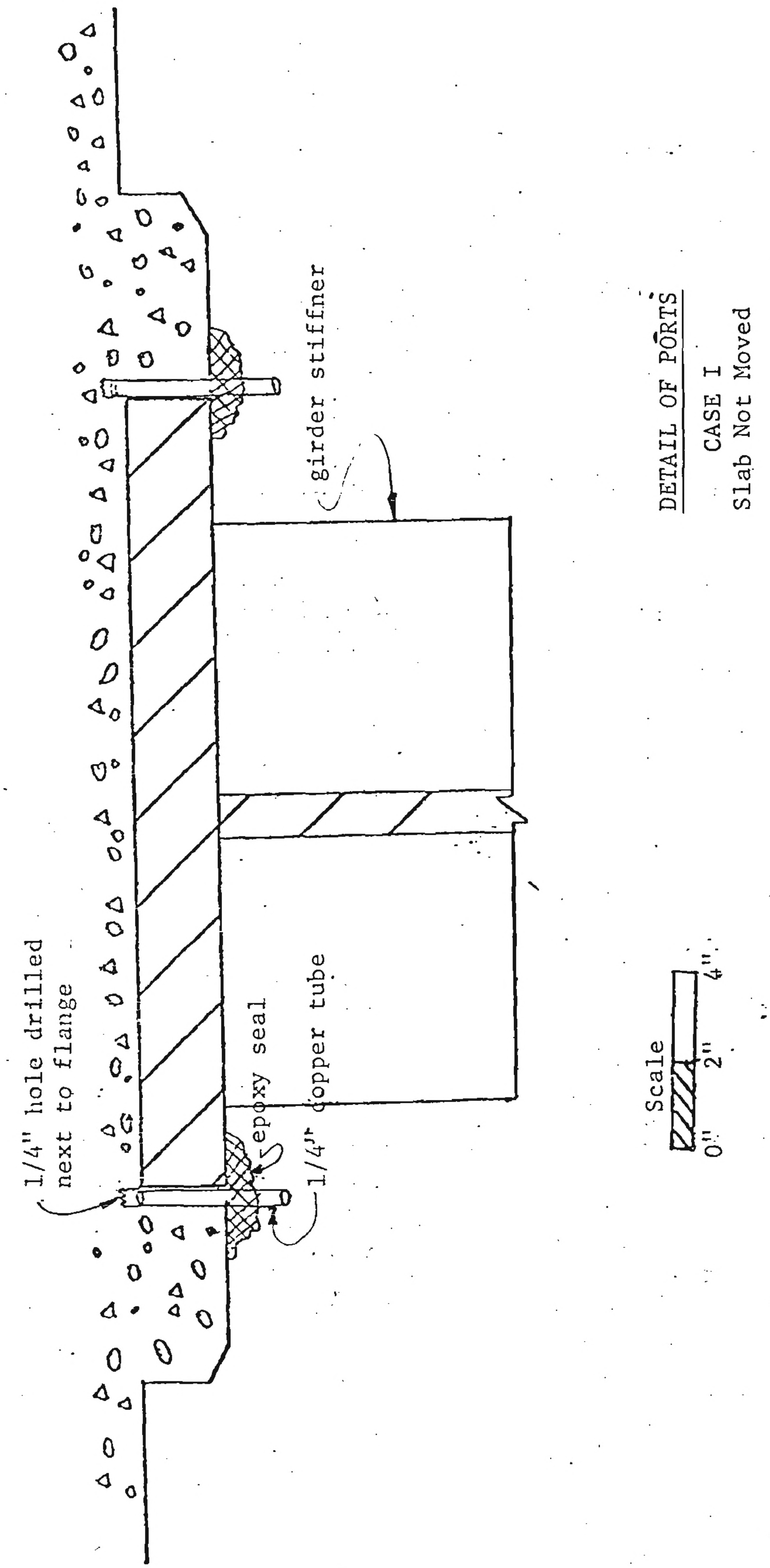


Figure 4. Port Installation, Girder 1

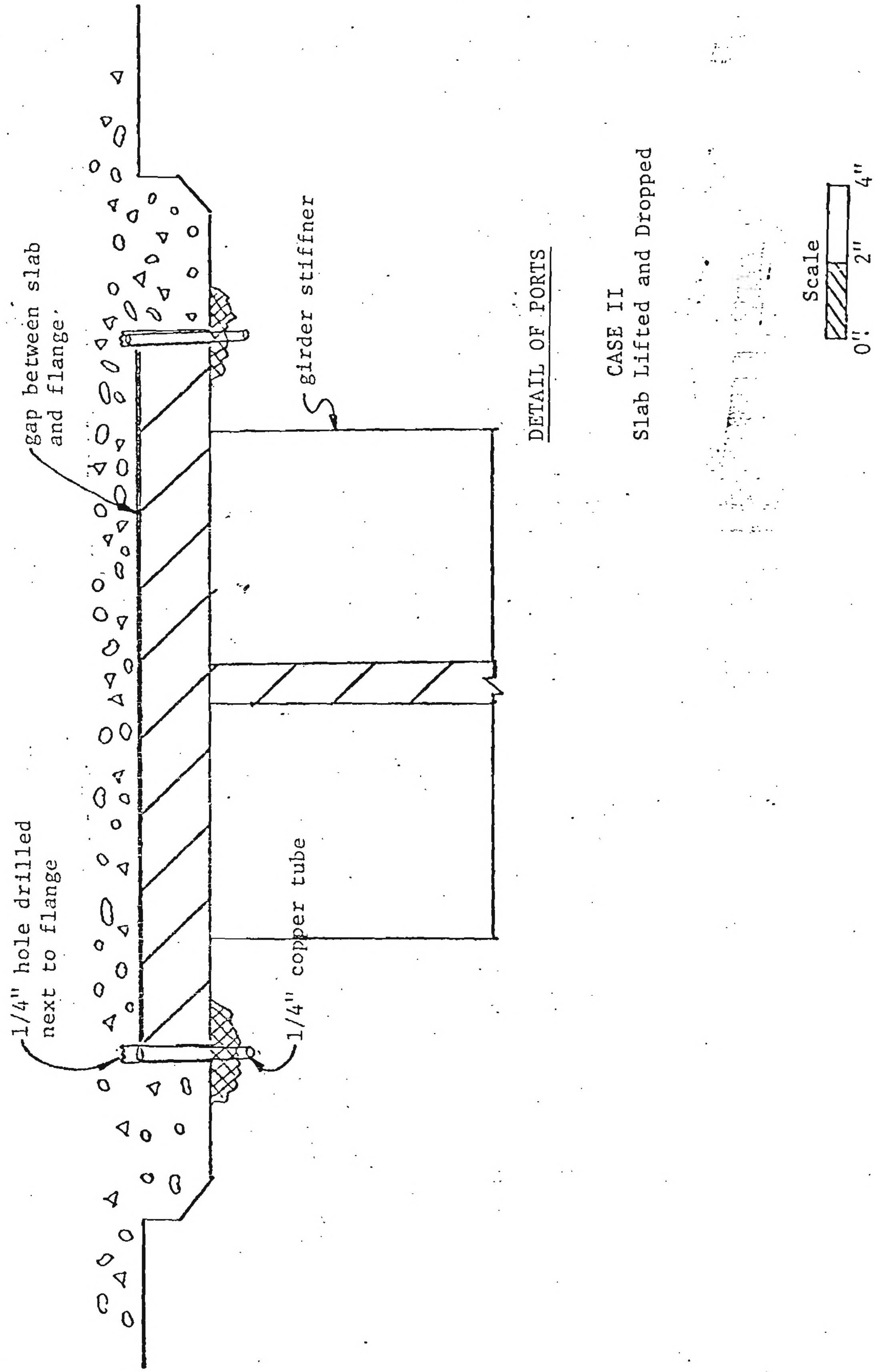


Figure 5. Port installation Girder 2

Figure 6: Sealer application around ports of Girder 1.

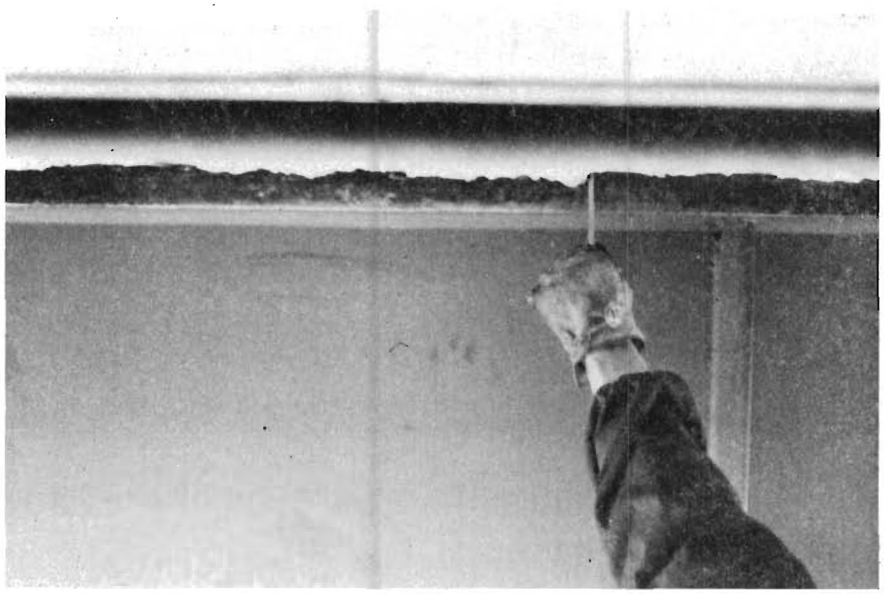


Figure 7: Insertion of copper tube ports into Girder 3.

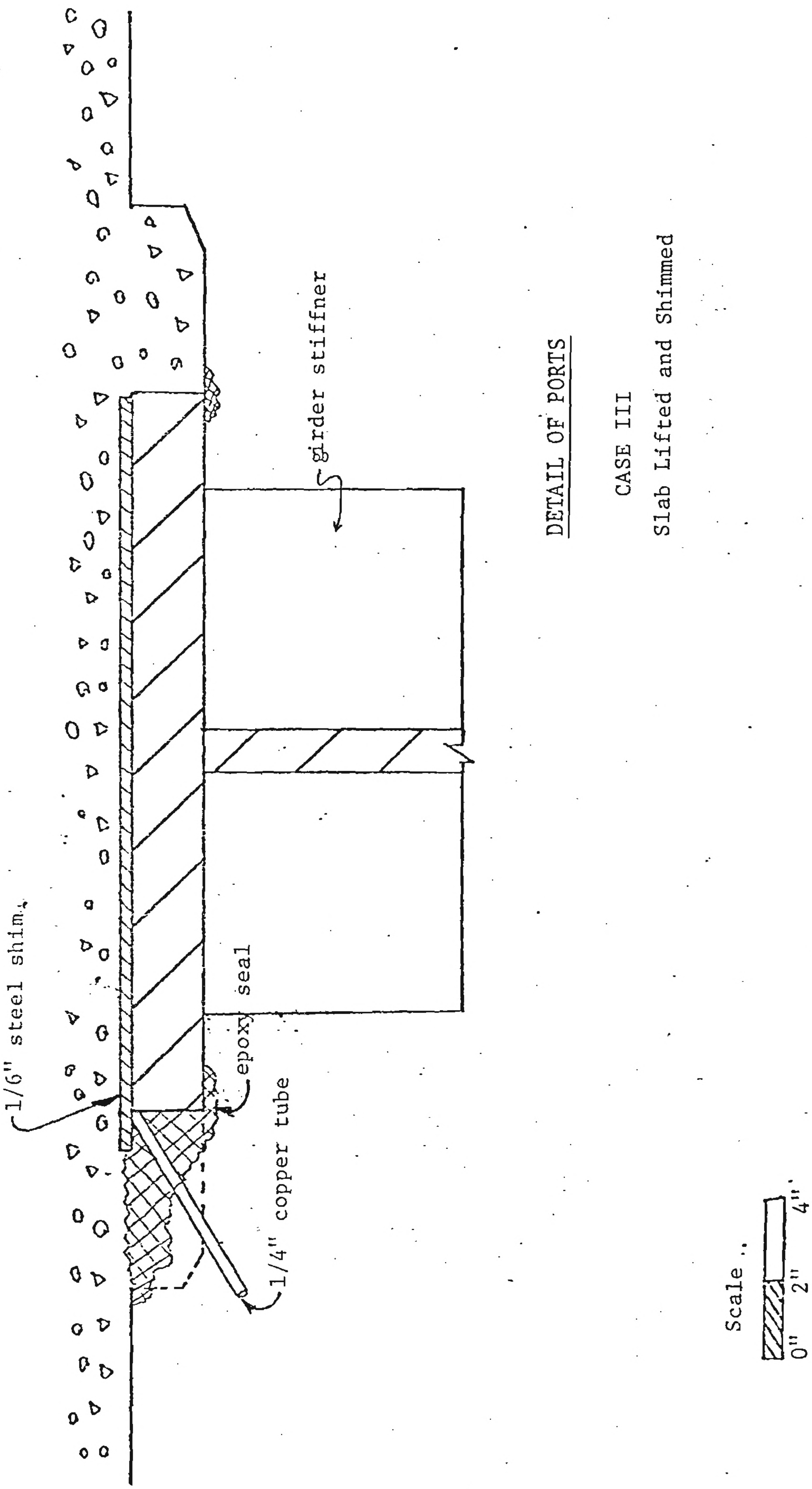


Figure 8 . Port installation, Girder 3

Figure 9: Applying Sealer

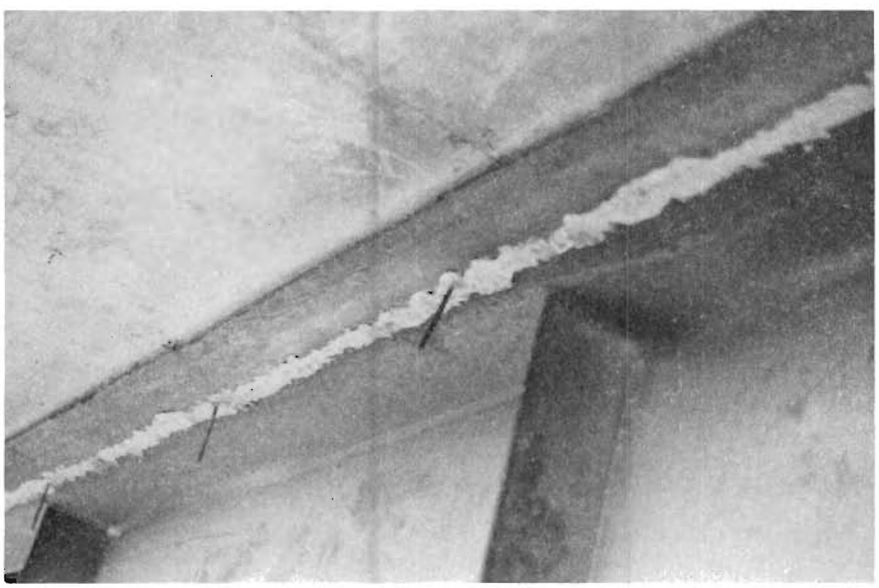


Figure 10: Girder after seal application

2.3.3 Sealer Testing

The adequacy of the sealer was tested using a freon gas procedure. The testing procedure involved pumping freon gas through one port while keeping all other ports taped shut. The air intake hose for a butane gas burner was passed around the sealed area. If a leak existed, the freon gas would be sucked into the hose and turn the flame from light blue to brilliant blue. Figure 11 shows this procedure. Each section of the seal was tested and marked if a leak was discovered; leaks were resealed.

2.3.4 Injection of Epoxy

Epoxy was injected on December 8 and 9, 1977. The type of epoxy used was Sikastix 350; this consisted of two components, a 100%-reactive modified epoxy resin and a reaction compound mixture. Penetryn System Inc., Restoration Department, of Willoughby, Ohio,* performed the epoxy injection using their patented injection gun and apparatus. Figures 12, 13 and 14 show the epoxy containers and injection gun with the pressure gages.

Weather conditions on December 8 and 9 were not favorable due to the low temperatures, averaging 36°F and 27°F respectively. A 120,000 BTU kerosene heater was mounted on the scaffolding and was used to heat the epoxy and injection machine so that the epoxy flowed satisfactorily (Figure 15).

Girder 3 was injected first at a maximum pressure of 20 psi. Only the ports on one side were used for injection. Flow of epoxy out of the single port on the opposite side indicated good flow across the flange. Even though cracks were sealed, epoxy leaks occurred on both sides of the stringer. Leaks during injection were sealed using a quick-setting hydraulic mortar.

*This service was obtained from a subcontract agreement.

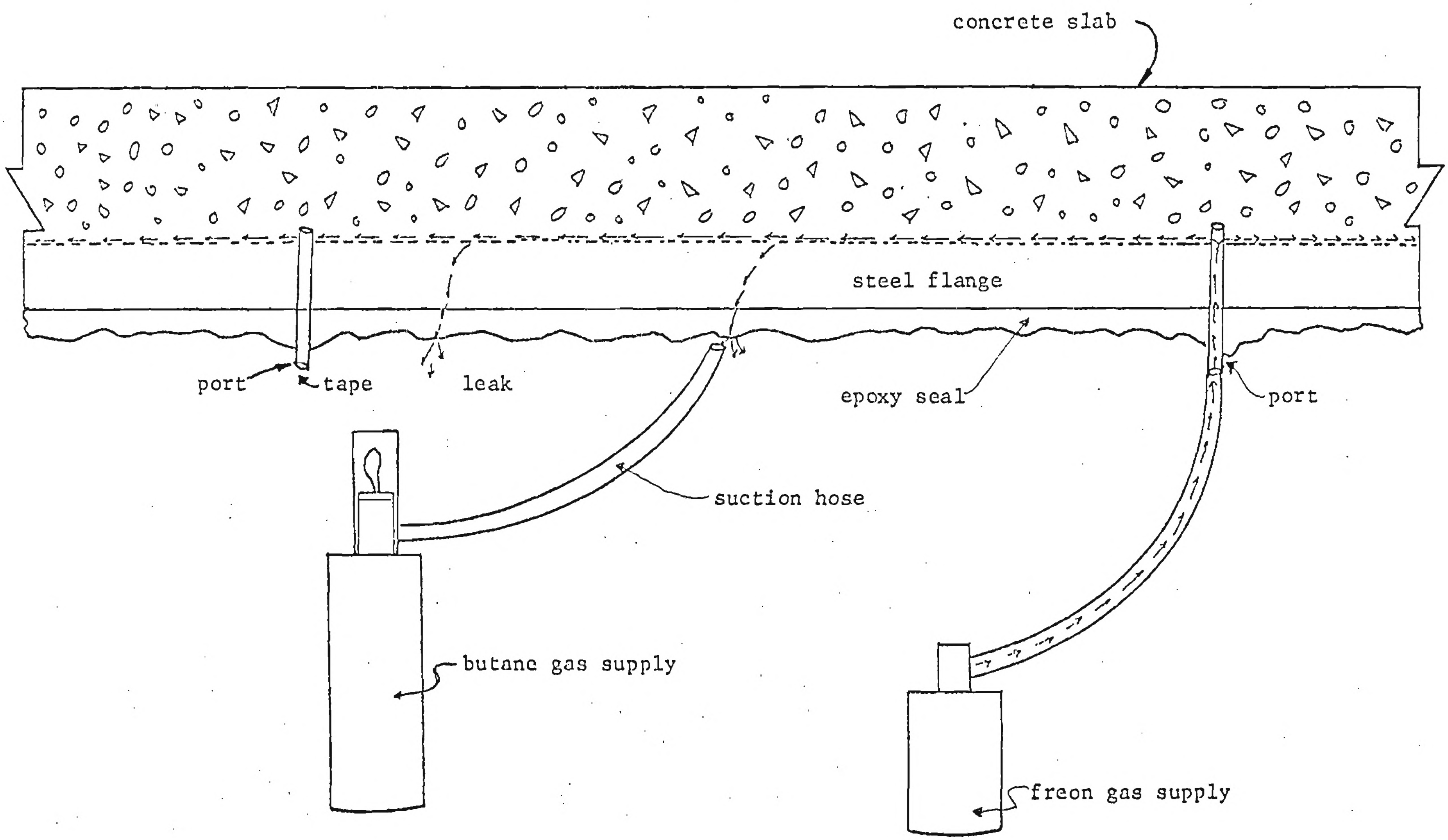


Figure 11: Seal testing procedure using freon gas and a butane tank with suction hose

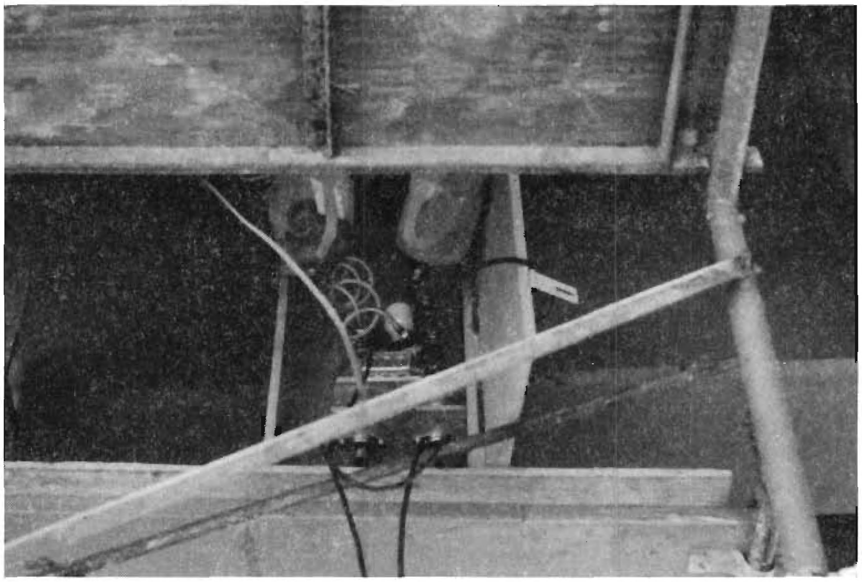


Figure 13: Pressure gauges on epoxy injection machine used to monitor flow.

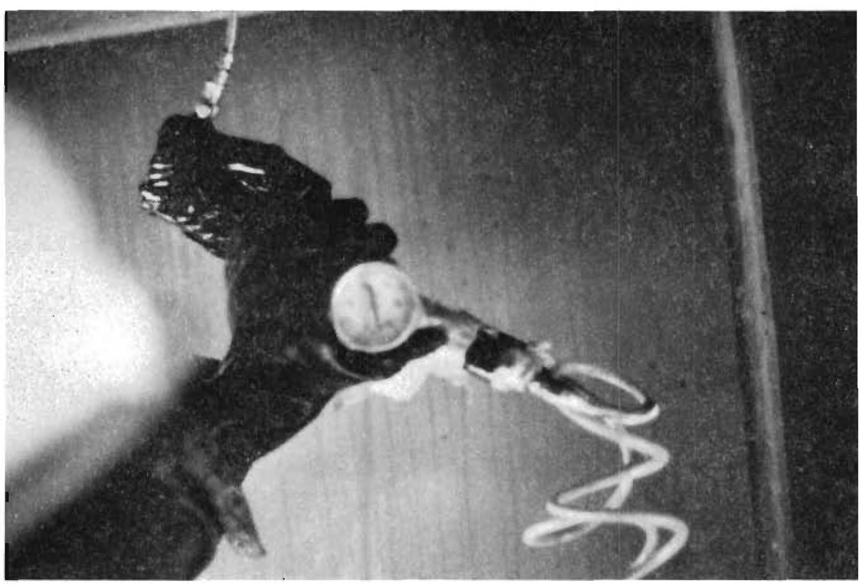


Figure 12: Epoxy injection machine.

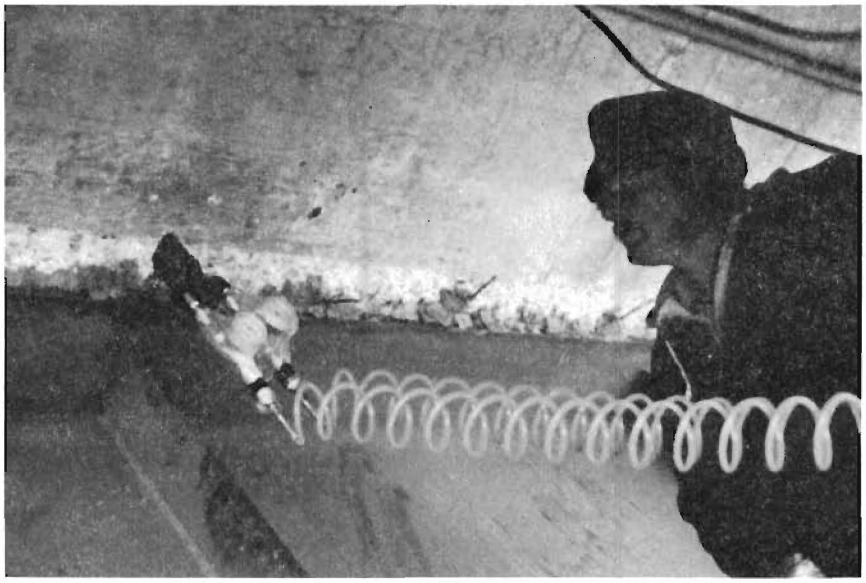


Figure 14: Automatic mixing at nozzle. Note separate hoses for epoxy components.

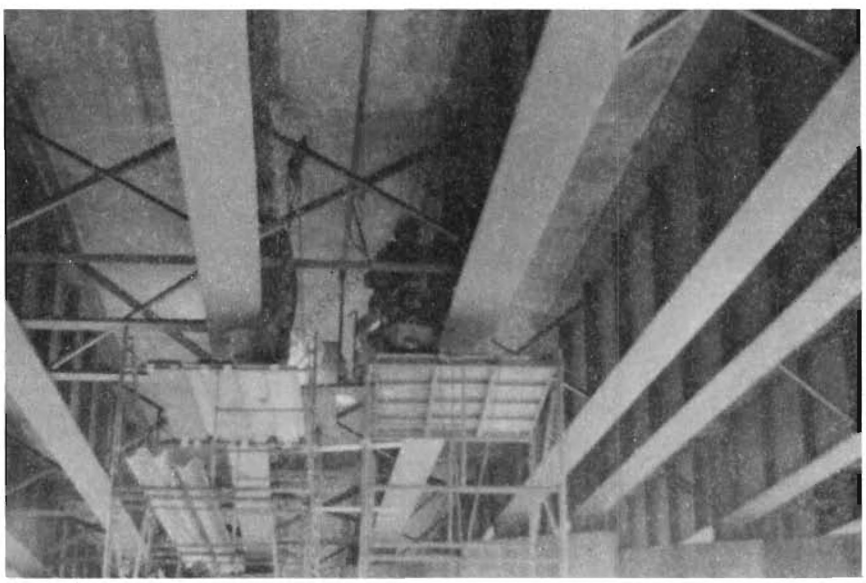


Figure 15: Heater used to keep epoxy viscosity low.

The procedure for injection was that the injection gun was attached to the port at the far west end of the gap and epoxy injected. When epoxy began flowing from the neighboring port, the first port was sealed closed by clamping and twisting the copper tube with pliers. The injection gun then was connected to this second port and injection resumed. This port-to-port procedure was followed until flow from the farthest port was observed, and that port clamped. Eleven quarts of Sikastix were used on this girder.

During injection of Girder 3, the pressure varied constantly. Twenty psi was first applied, and as epoxy moved into the gap the pressure dropped to zero. The 20 psi then was reapplied to continue the injection. Several pressure bursts were used at each port.

Girder 2 was injected next. A maximum pressure of 120 psi was required for injection of Girder 2, which had been lifted and lowered. The five ports on the north side of the girder were injected first using the port-to-port procedure. The next day (December 9) injection was attempted on the ports on the south side. Some of the south ports were filled with solid epoxy and, therefore, could not be injected. The filling of the south ports indicated that epoxy had flowed across the entire flange. Approximately 2-1/2 quarts of epoxy were used for this girder.

The significant difference in injection pressure and amount of epoxy between these two girders clearly illustrated that the shimmed section was easier to inject. Mr. Roy Pollock (14) of Penetryn stated that the typical maximum injection pressure for other applications is 40 psi. The 120 psi needed for Girder 2 implied that the gap between the slab and girder was very small.

Girder 1 was injected last. All 10 ports on both sides were used for injection with a maximum pressure of 200 psi. No flow was observed.

2.3.5 Curing

Curing time as stated in the technical specifications for Sikastix 350 is three days at 73°F. Curing time on this project was indeterminate due to the cold weather. A record of the daily temperatures is in the Appendix C Table C-1. It was estimated from previous experience (14) that approximately two weeks at temperatures above 55°F would cure the epoxy. As the temperature record indicates, several weeks of above 50°F occurred before stripping of the deck from the girders.

Samples of the injected epoxy indicated complete hardening before the deck removal.

2.4 Removal of the Slab

2.4.1 Deck Sawing

Visual inspection of the girder-to-slab bond required removal of the slab from the stringer. To ease this removal the slab was cut into sections averaging 1 ft x 4 ft as illustrated in Figure 16. Figure 17 shows the same cuts from the bottom of the slab. Cutting was conducted by Georgia DOT personnel using a diamond saw. The extensive reinforcing in the slab made cutting difficult. Two 1-1/2-in diameter holes were bored through each section to facilitate lifting of each section. This can also be viewed in Figures 16 and 17.

2.4.2 Georgia DOT Method

The first method used to remove the cut sections was designed by the Georgia Department of Transportation Research Group. As shown in Figures 18, 19 and 20, the method used a 50-ton hydraulic jack to lift a cross-beam which was bolted to the sections. Unequal forces were developed on either side of the girder which resulted in shear type failure of the sections as shown in Figures 21 and 22. The portion of the sections directly over the stringer remained bonded to

Figure 16: Saw cut slab over Girder 1.

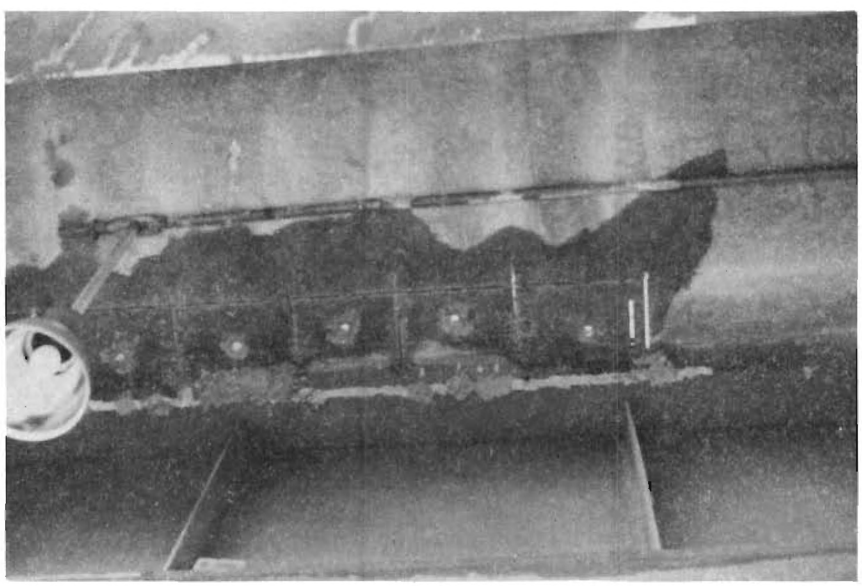


Figure 17: Saw cut clab over Girder 1, view from under bridge.

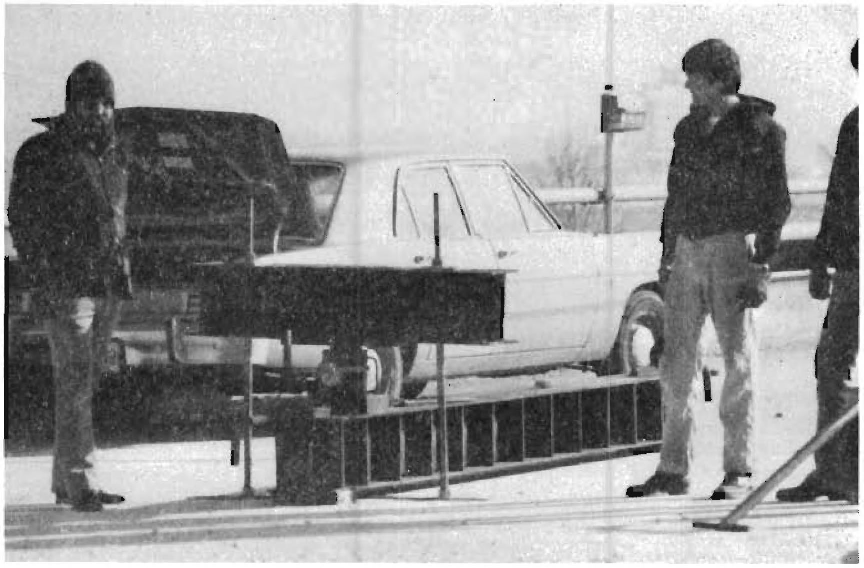


Figure 18: GDOT slab lifting system.

Figure 19: GDOT slab lifting system - measuring deflection at failure

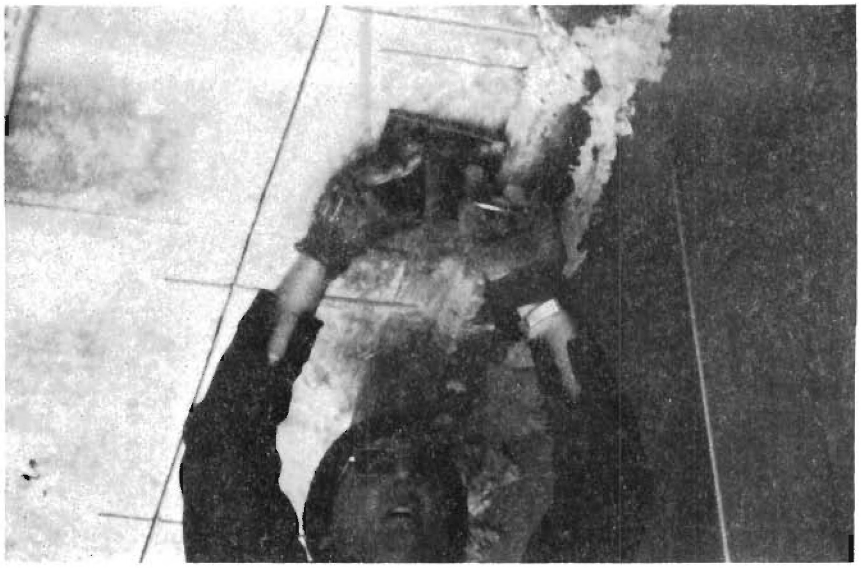


Figure 20: Installation of bottom load plate

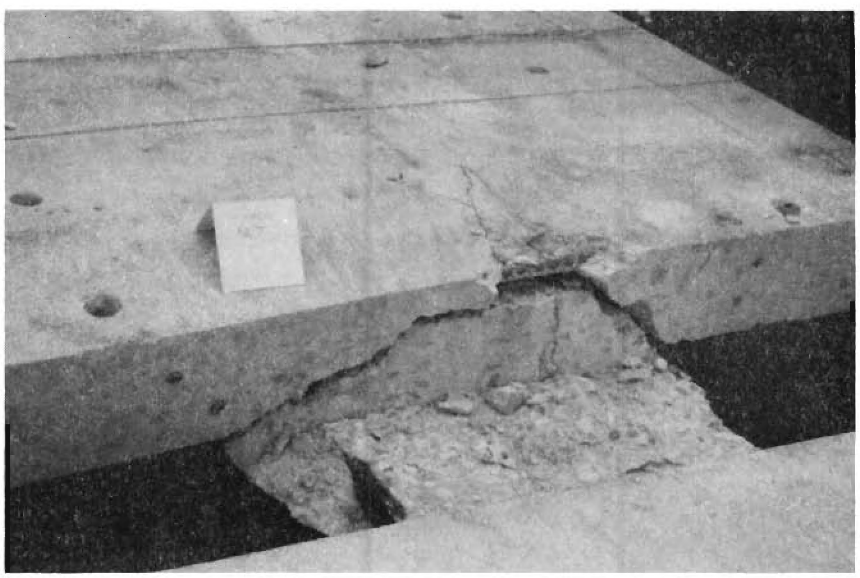


Figure 21: Shear type failure resulting from unbalance of moments. View from on top of deck slab.

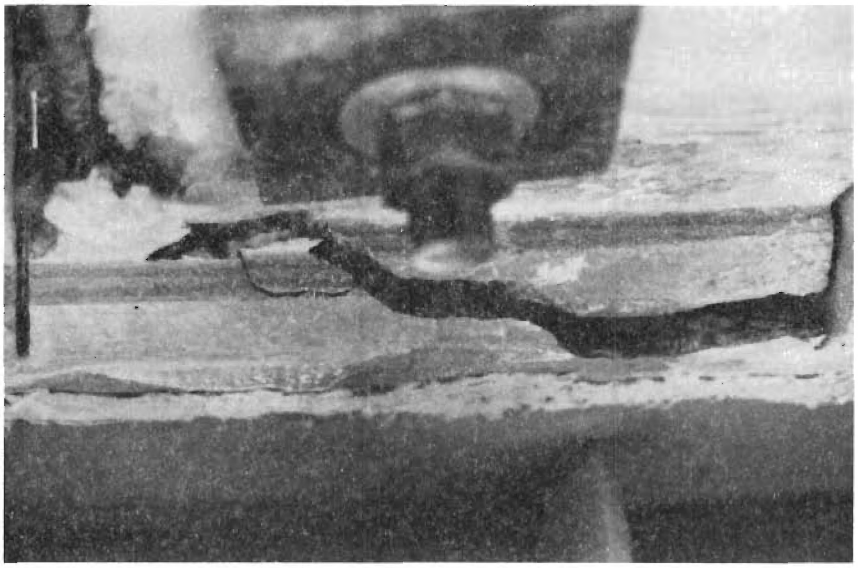


Figure 22: Shear failure of slab. View from bottom of slab.

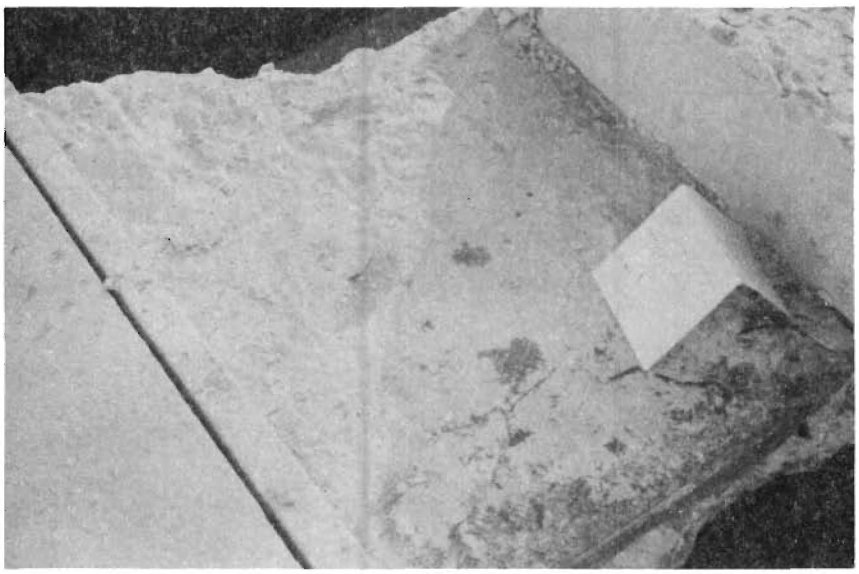


Figure 23: View of flange top after slab removal. Girder 1.

the flange which made visual inspection impossible. Load readings provided no indication of the bond strength of the epoxy.

2.4.3 Georgia Tech Method

A second lifting method was developed which used the same jacking arrangement that was used to lift the uncut slab previously. Hydraulic jacks were placed on each side of a girder as is shown in Figure 3. Each jack was again loaded at the same rate until the section popped up, and the loads fell. The section was then lifted out of place by four men. Visual inspection of the bottom of the slab and of the top of the stringer followed. Figures 23 through 33 show various sections. Detailed descriptions are included under Section 3.1.

The jacks were moved to different locations to lift the many sections. A sufficient number of sections on each of the three stringers was removed to thoroughly view the bond surface. A tabulation of the loads experienced is included in Table 1.

Where large areas of concrete remained on the girders, a jackhammer was used to chip the concrete so that the slab-girder interface could be viewed.

2.4.4 Push-Off Tests

Push-off tests were designed to provide better quantitative data on the bond strength of the concrete-to-steel connection for each stringer. As shown in Figures 34 and 35, hydraulic jacks were positioned horizontally to push against the side of a section and thereby produce a direct shearing force along the slab-girder interface.

The entire deck of the bridge was used as the reaction for the push-off jacks. The jack was located as close to the flange as possible in order to minimize overturning moment in the push-off section; the center of the jacking force was about 3 in above the flange. A ball-bearing swivel head bearing plate was used to compensate for the difference in angle between the saw cuts

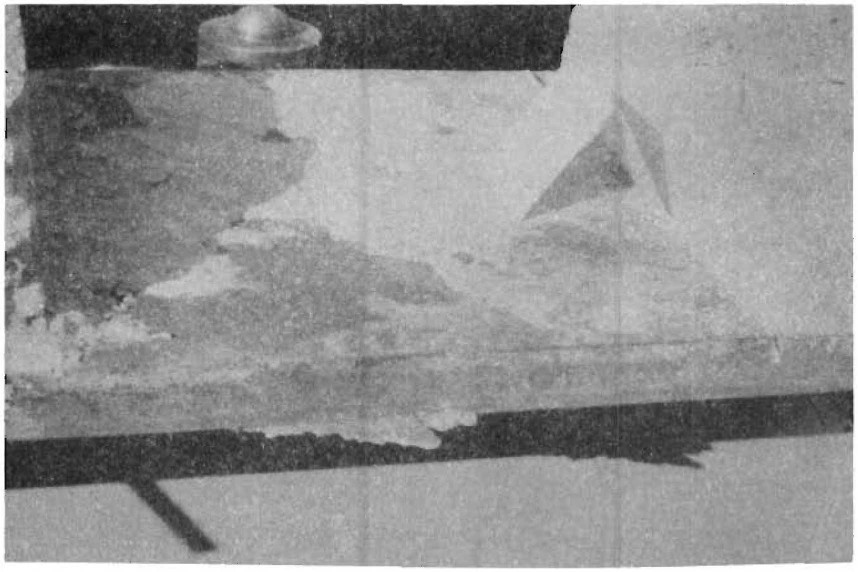


Figure 24: View of flange top after slab removal, Girder 1.

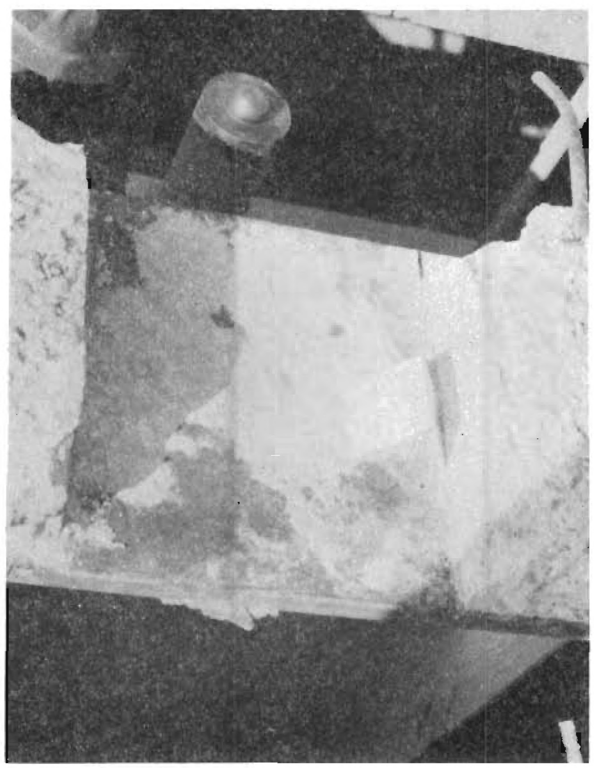


Figure 25: View of flange top after slab removal, Girder 2.

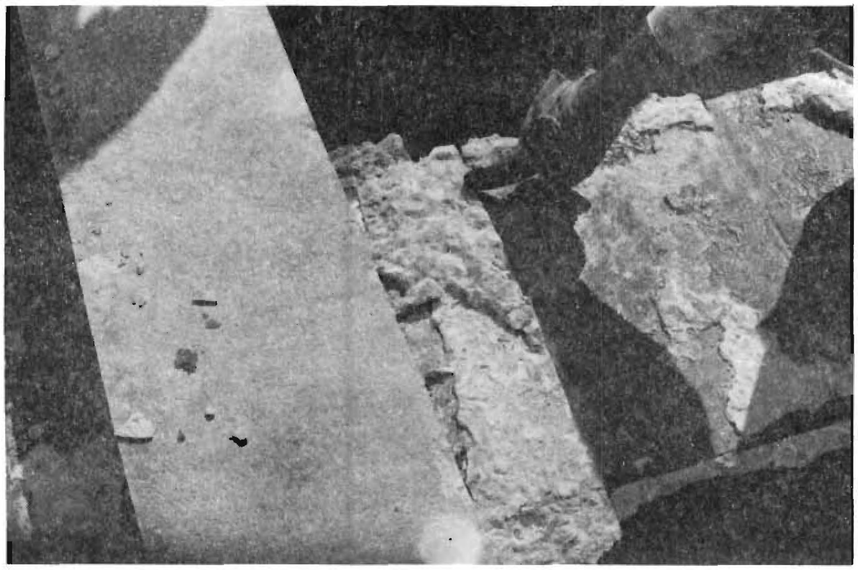


Figure 26: View of flange top after slab removal, Girder 3.

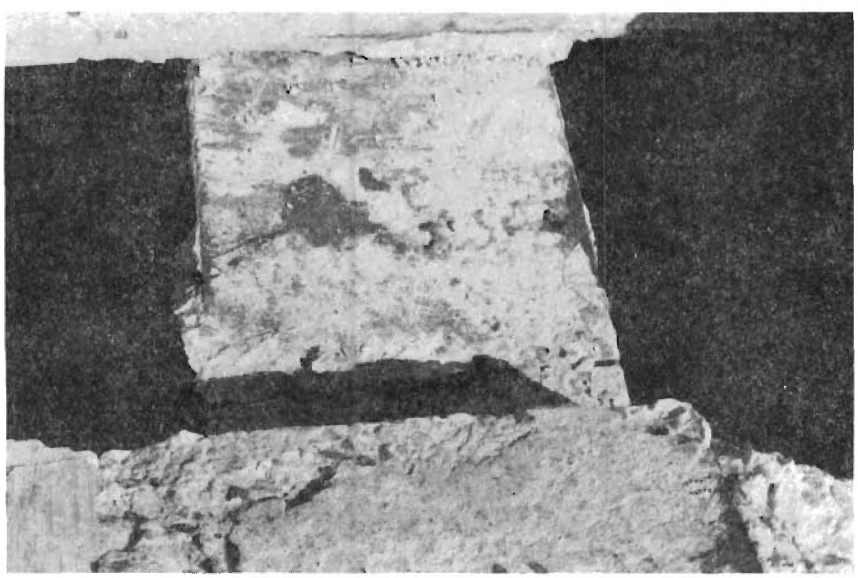


Figure 27: View of flange top after slab removal, Girder 3.

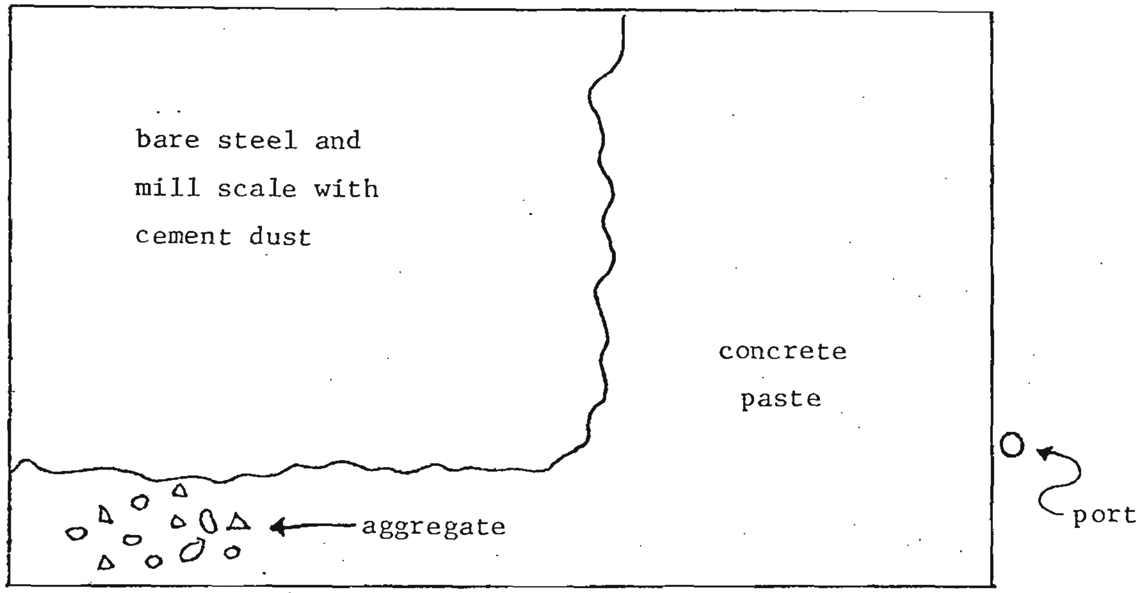


Figure 28: Girder 1, Section 4

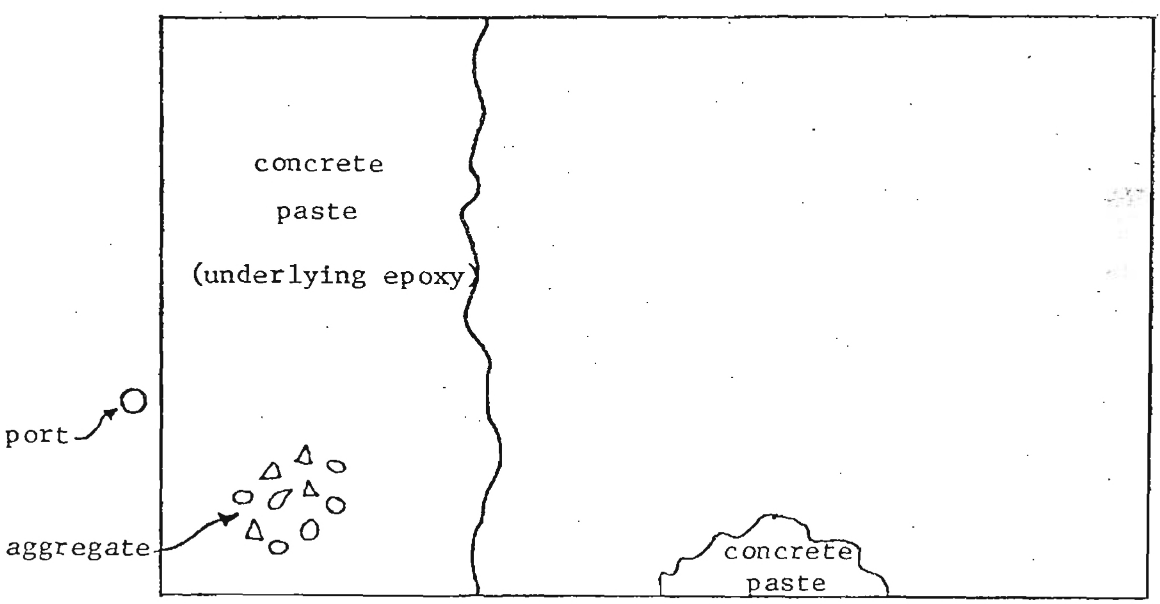


Figure 29: Girder 2, Section 1

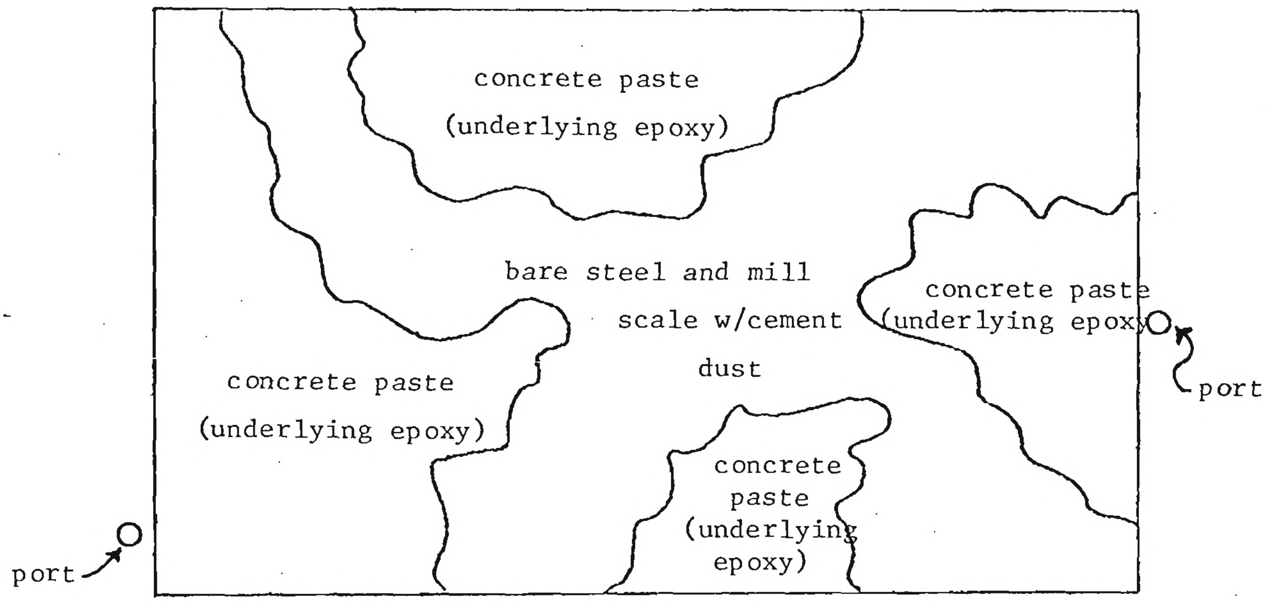


Figure 30: Girder 2 Section 2

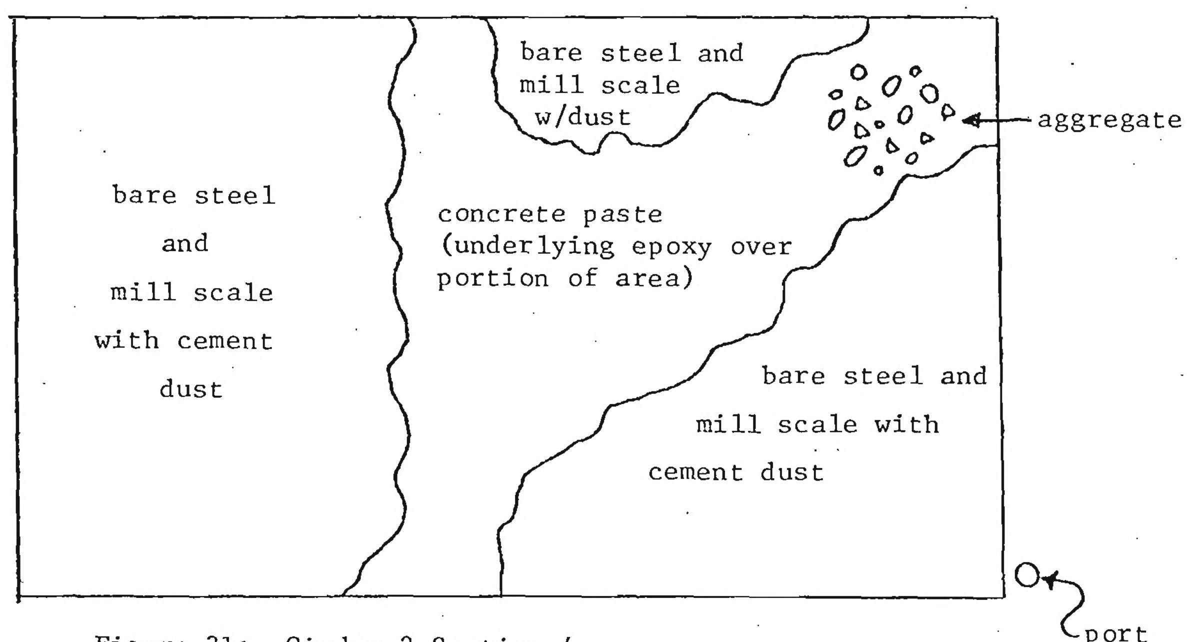


Figure 31: Girder 2 Section 4

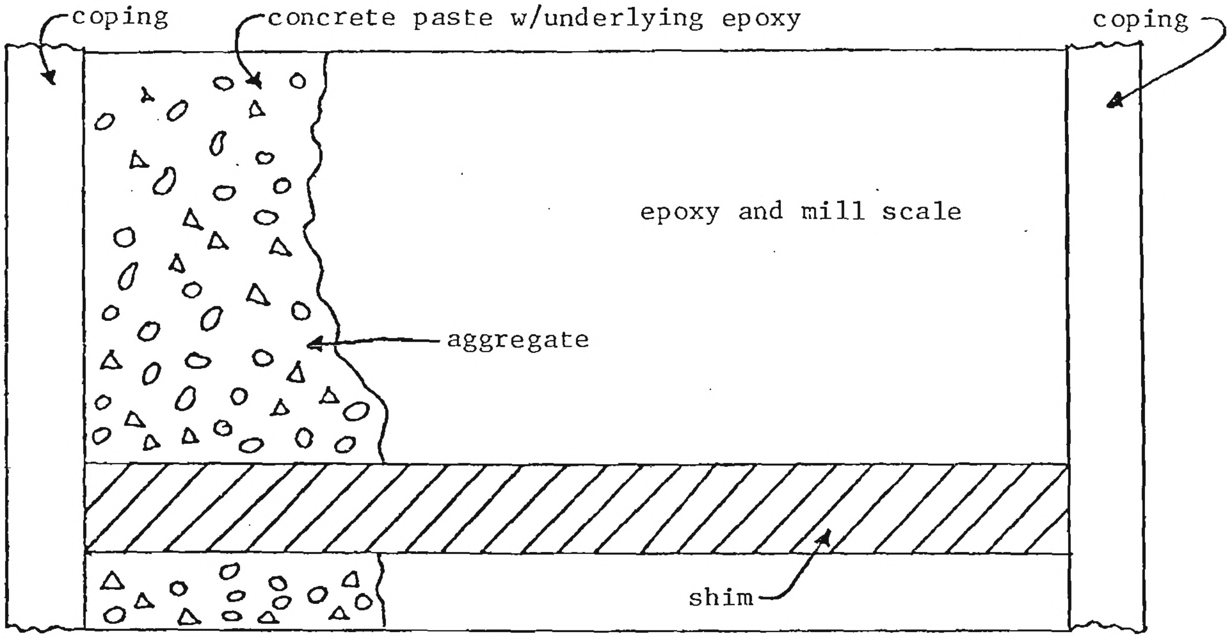


Figure 32: Girder 3 Section 1

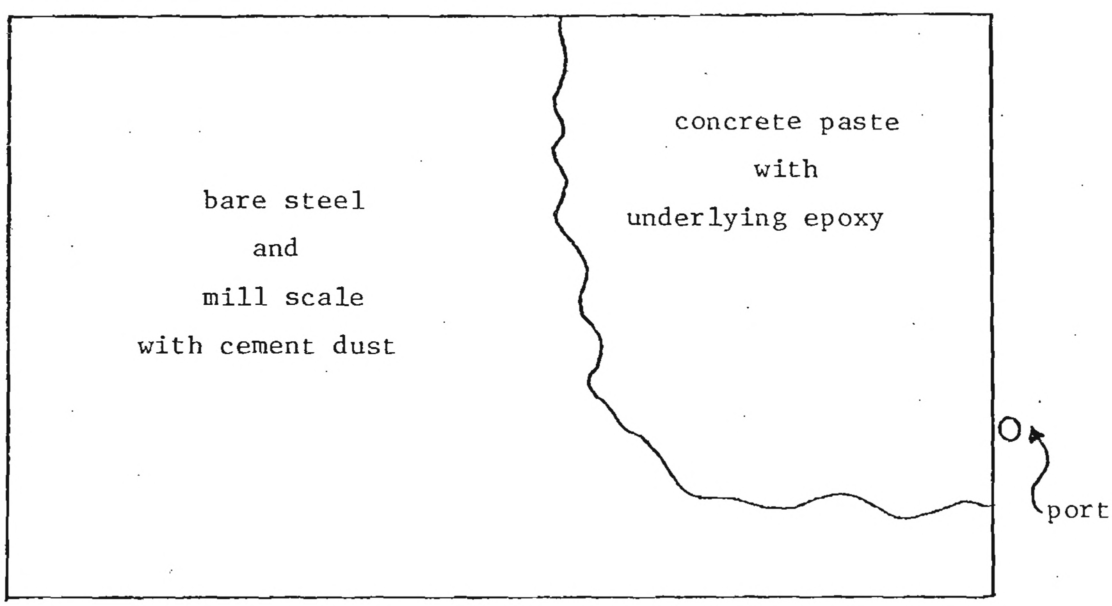


Figure 33 : Girder 3 Section 4

TABLE 1

LOADS FOR LIFT TEST

TYPE			
BEAM #1	1	50	20
BEAM #1	2	4.5	4
BEAM #2	2	2.25	2.5
BEAM #2	6	1	1
BEAM #2	7	1.25	1.25
BEAM #3	3	3.75	2.75
BEAM #3	4	3.0	2.5

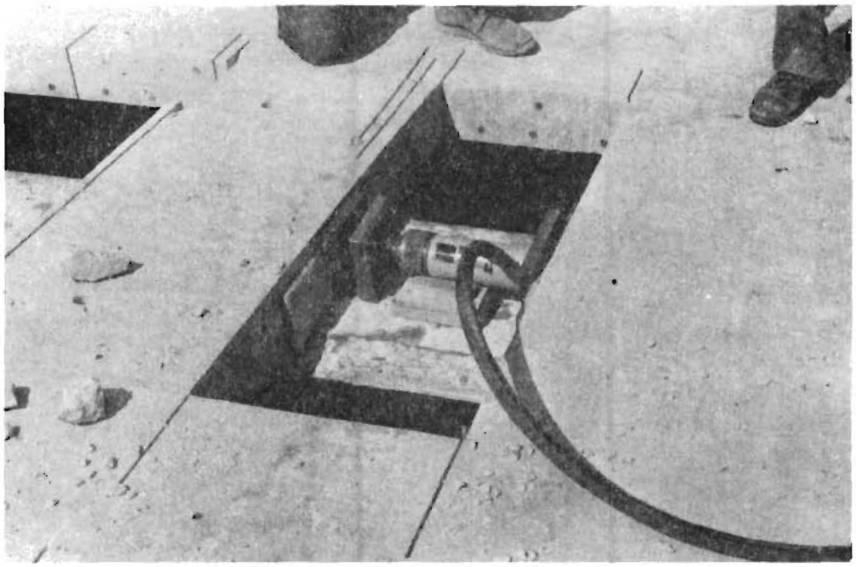


Figure 34: Push-off test using one jack.

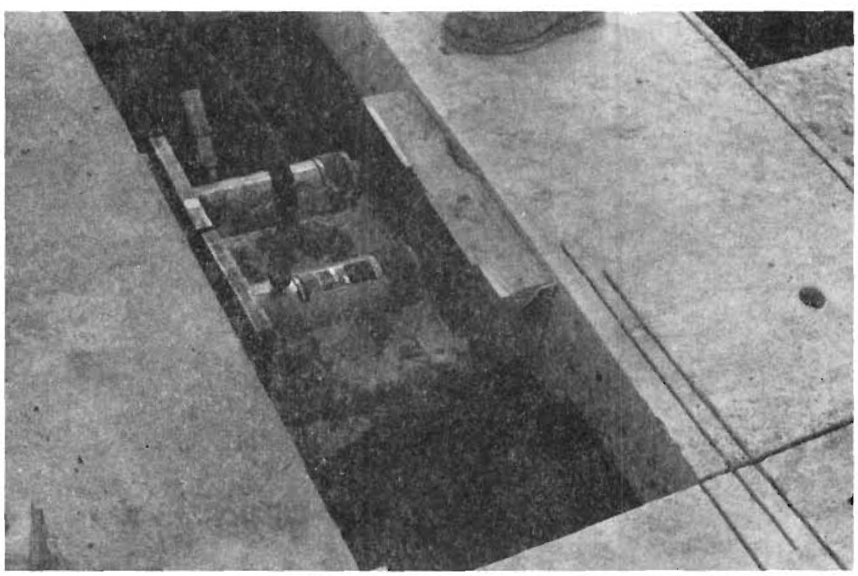


Figure 35: Push-off test using two jacks.

of the various sections. Timber spacers were used to transfer the compression from the jack to the push-off section (Figure 36).

The push-off tests were conducted by pressurizing the jacks until the section "popped" and slid along the girder. When two jacks were used, equal force was maintained in each jack, and the maximum pressure occurring just before failure was recorded. When bond failure occurred, the jacking pressure decreased instantly to a low value.

Visual inspection of the samples was made after removal of the sections. The actual area of epoxy bond was estimated and compared to the overall area of the section. The push-off tests concluded Phase I field work.

3.0 RESULTS OF EXPERIMENTAL INVESTIGATION

3.1 Visual Observations

The visual inspection of the top surface of the three girders yielded the following observations. Figures 28 through 33 illustrate the observations of the flange surfaces for the three girders.

3.1.1 Girder 1

No epoxy was observed between the flange and the slab of Girder 1. This observation confirmed that made during epoxy injection which was that no epoxy flowed into the joint where the slab had not been raised. Cement paste adhered to about 50 percent of the flange area and indicated good cement-to-steel bond strength. Intensive hammering with a chisel was needed to remove the paste. Mill scale and rust particles were observed on the underside of the sections which were removed.

3.1.2 Girder 2

Epoxy covered around 30% of the flange where the slab was lifted and lowered prior to injection. The amount of epoxy near ports was large, which

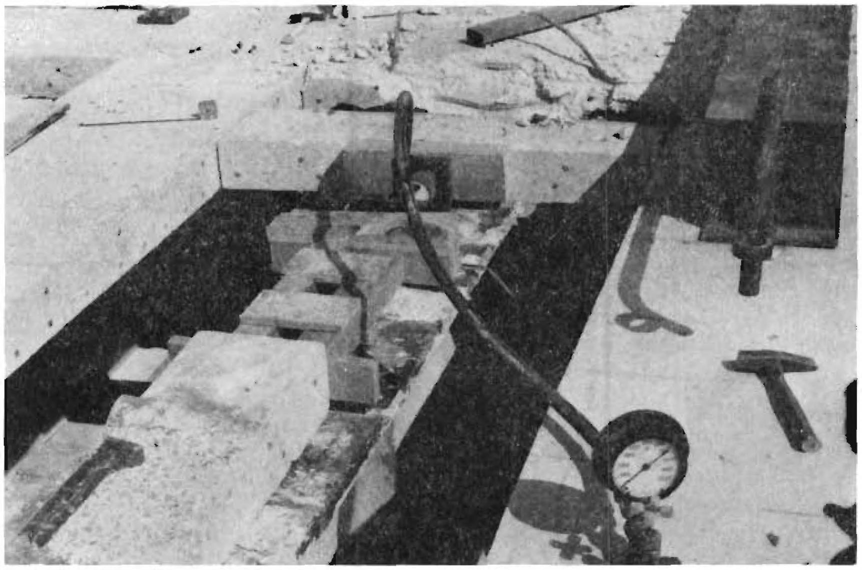


Figure 36. Timber and concrete spacers for push-off tests.

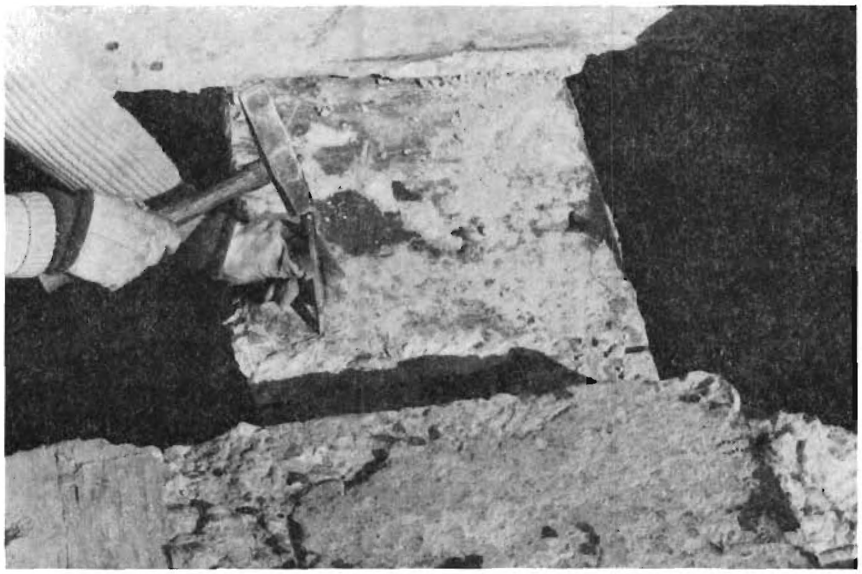


Figure 37. Chipping cement paste to locate epoxy.

indicated that flow was good near ports. Epoxy flow was continuous across the width of the flange in only a few locations.

Mill scale, rust, and cement paste covered about 50% of the area. As stated above, chipping at the paste with a hammer and chisel (Figure 37) revealed that epoxy was found under a portion of the paste. Under some areas of the paste, no epoxy was found.

3.1.3 Girder 3

Epoxy covered around 60% of the flange where the slab was raised and shimmed prior to injection. The thickness of the epoxy was uniform over the entire flange width. Epoxy was clearly visible on the top flange and also on the bottom of the slab.

Mill scale and cement paste covered the remaining portion of the flange. Epoxy has bonded to this weaker material as shown in Figure 38. Bond failed between the scale-paste material and the steel.

3.2 Quantitative Measurements

The push-off tests provided good quantitative measurements of the shear-bond strength between the slab and girders. Results of these tests are given in Table 2. Nominal push-off stresses were calculated by dividing the pushoff force by the length of the section, and the width of the flange. The push-off results indicated that the natural adhesion of Girder 1 was about the same or greater than the epoxy bond of Girders 2 and 3.

For Girder 3, a "net" stress was calculated by dividing the total push-off force by the area covered by epoxy. The net stress implied that the epoxy transferred all the shear force. The net stress was nearly twice the nominal stress for Girder 3 and was greater than the nominal stress of the other girders. Comparison of the three cases indicates that the net epoxy bond is greater than the natural concrete-to-steel bond.

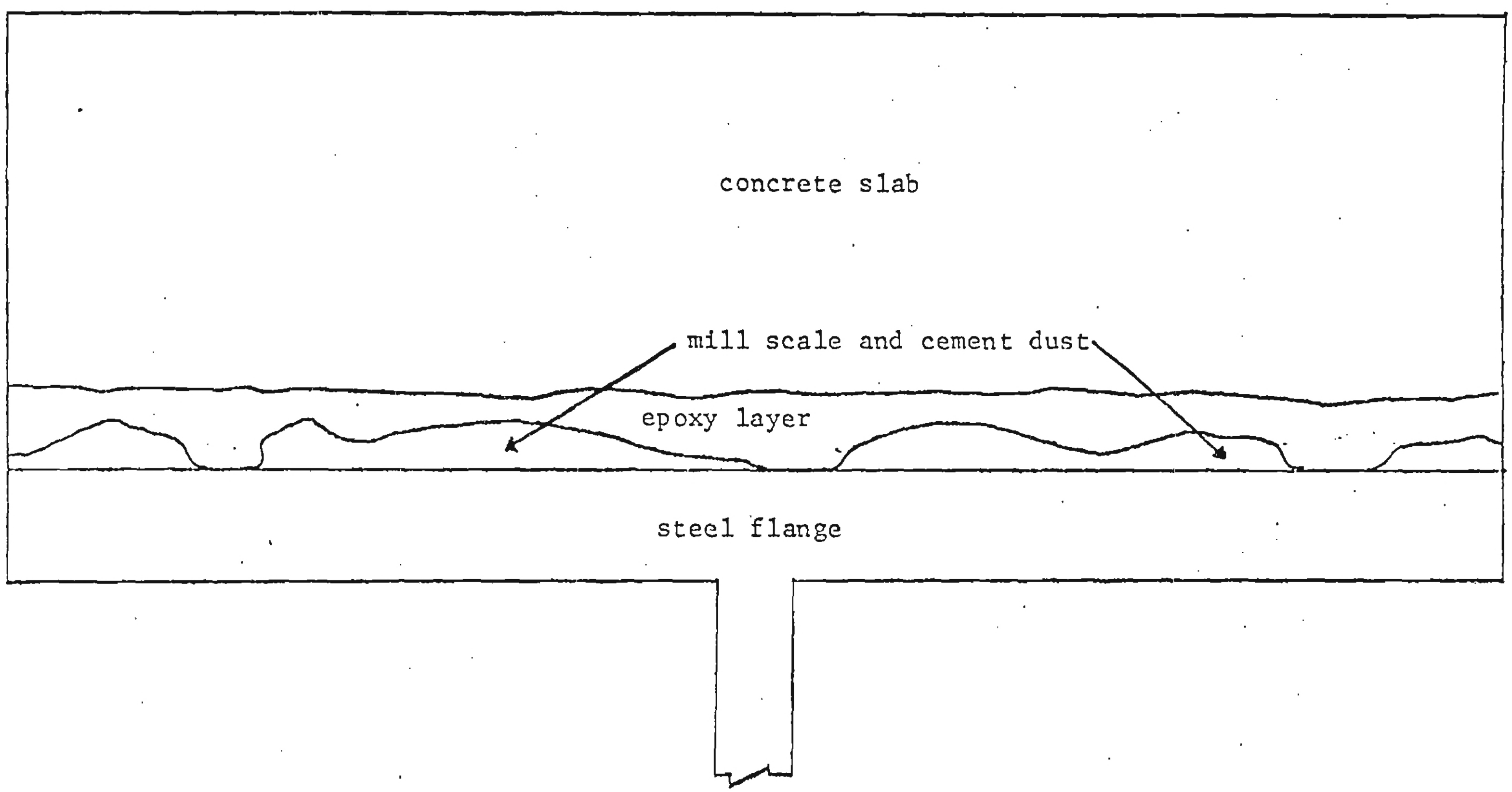


Figure 38 : Bonding condition encountered in Beam #2 and Beam #3. Epoxy bonded to mill scale and cement dust.

Table 2. PUSH-OFF TEST RESULTS

Girder	Push-off Force(lbs.)	Gross Stress(in^2)	Gross Stress(psi)	Net Area(in^2)	Net Stress(psi)
1	55,500	390.96	142		
2	45,375	367.2	123.6		
2	27,000	179.3	150.6		
3	23,000	198.7	115.7	95.7	233
3	16,875	183.6	91.9	98.4	171.5

4.0 ANALYTICAL INVESTIGATION

4.1 Finite Element Analysis of Push-Off Sections

A finite element analysis was made of a push-off section to better understand the bond stress conditions at failure, particularly for the Girder 1 section. The physical model used for the analysis was Section 6 of Girder 1 as illustrated in Figure 39. The coping was ignored because it was cracked. The total load applied to the model was the 55,500 lb. failure load of the section.

The bond stress distribution for lines 1, 2, and 3 (Figure 39) is shown in Figure 40 where the distance is measured from the loaded face along the length of the girder toward the back face of the section. The bond stresses vary from a maximum of 756 psi at the loaded face to 18 psi at the back face. The average stress across the loaded face was 476 psi; the nominal stress for the entire bond area was 142 psi, the same for the analysis and the push-off test.

Compared to the nominal stress of 142 psi, the maximum stress at failure was over five times greater, and the average loaded-face stress was over three times greater. These higher stresses mean that the natural bond between the cement and steel of Section 6, Girder 1, did not fail when a stress reached 142 psi, but that the failure was initiated by significantly greater stresses at the loaded face. Therefore, in considering the bond strength of the deck-to-girder connection, the maximum stresses are most important whereas the nominal stresses give only an indication of the bond strength.

While finite element analyses were not conducted for each push-off section separately, it may be deduced that the maximum bond stresses and average loaded face stresses at failure were about five times and three times the nominal stresses, respectively.

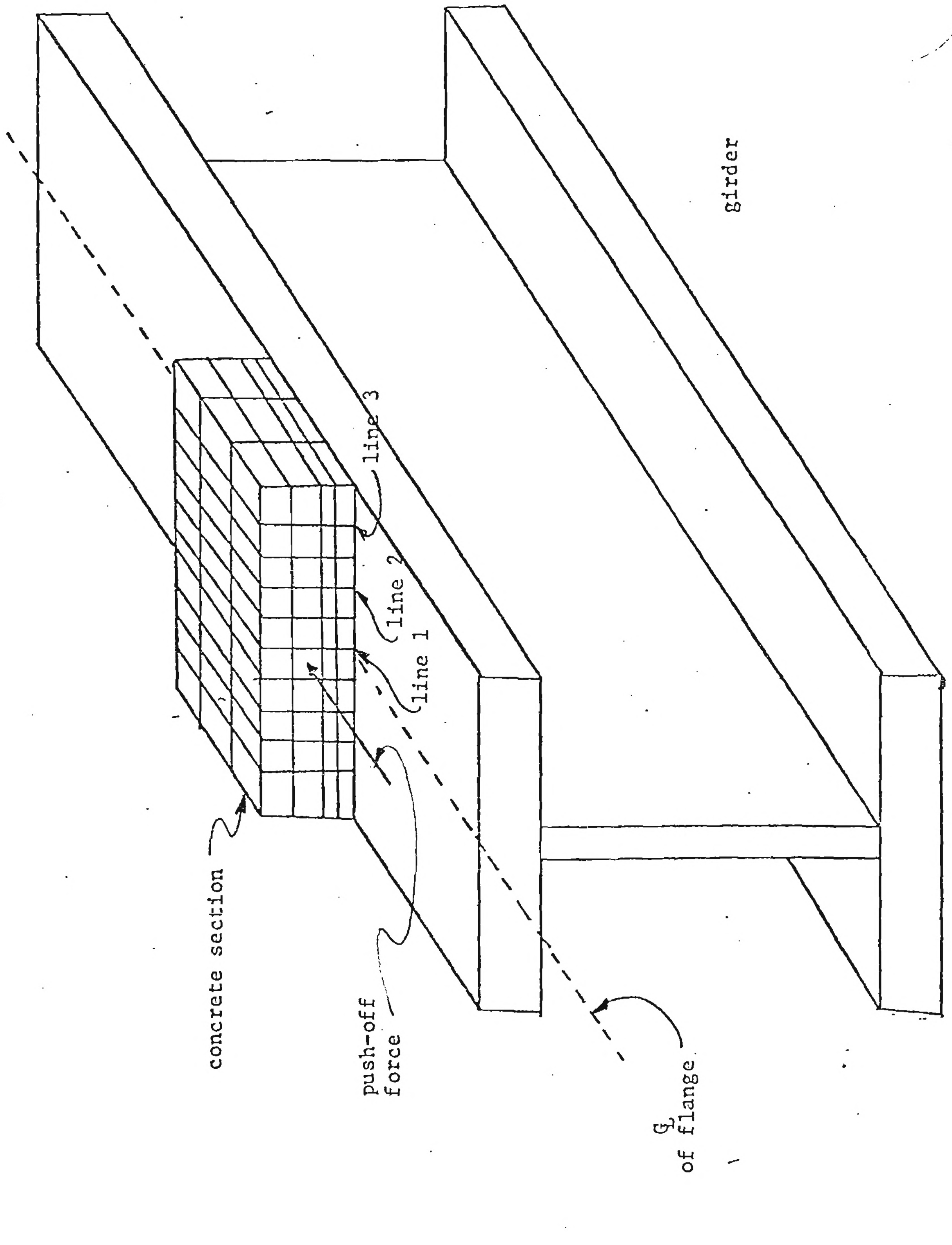
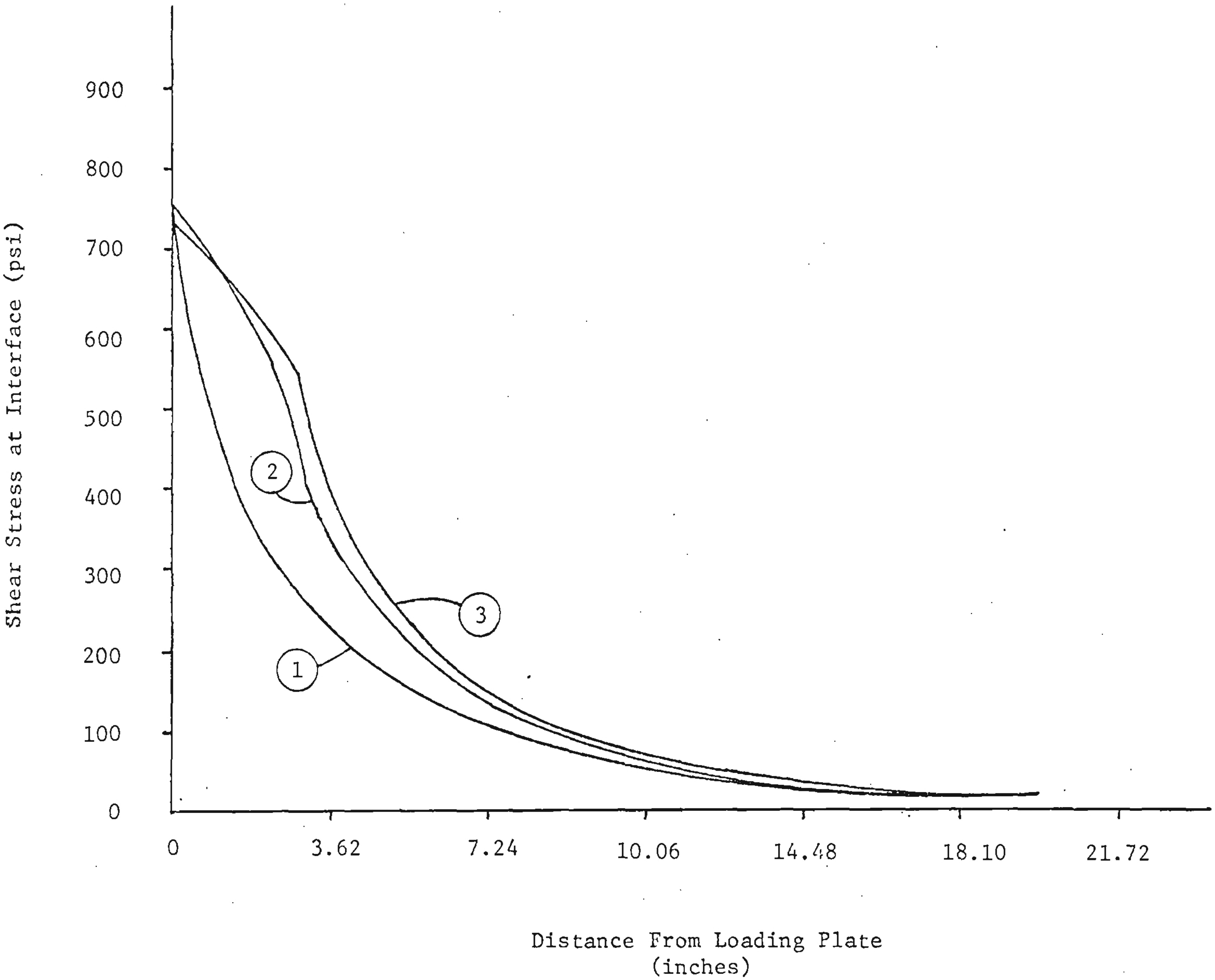


Figure 39 : Finite element model of push-off tests

Figure 40. Distance From Load Plane vs Shear Stress



In a bridge structure, the shear-bond stress at the deck-to-girder interface may not resemble that given by the previous finite element analysis. Rather, the stresses would be more uniform; the very high loaded face stress would not be present. The more uniform stress state would imply that the average loaded face stress, approximately 400 to 500 psi, would cause the shear-bond failure. Further analytical and experimental research would be needed to accurately specify these failure stress values.

4.2 Increase in Section Modulus

The section modulus of the steel stringers or of the stringer-deck composite section is directly related to the moment capacity and load carrying ability of a bridge structure. The section modulus of a non-composite girder-slab structure may be increased by making the two components composite.

The extent of the increase was determined for a variety of standard rolled steel sections together with concrete slabs of varying thicknesses and concrete strengths. The width of the slab was taken as 12 times the thickness as specified by AASHTO (1). The ratios of composite to non-composite section modulus are given in Table 3. The ratios vary from a low of 1.2 for a W36x300 section to a high of 1.6 for a W27x84 section. The trend demonstrated in Table 3 is that the section modulus increases more for smaller steel sections and for higher strength concrete. The section modulus based on the stress at the bottom fiber of the steel member is most important because that stress determines the allowable capacity of the member.

These ratios clearly show that significant load capacity increases may be attained if the non-composite sections were made composite. For shorter existing bridges where the steel stringers are small, the capacity increases would be the greatest for a non-composite to composite conversion.

Table 3. Section Moduli for Composite Structures

Section Type	Slab Thk. (ins)	Average Ratio Composite Section/Section Modulus					
		Bottom			Top		
		Concrete Strength					
		5000 (psi)	4000 (psi)	3000 (psi)	5000 (psi)	4000 (psi)	3000 (psi)
W36x300	6	1.227	1.214	1.203	2.108	1.949	1.824
	7	1.271	1.257	1.245	2.395	2.205	2.054
	8	1.311	1.297	1.285	2.636	2.422	2.250
W36x280	6	1.240	1.228	1.216	2.206	2.037	1.904
	7	1.285	1.271	1.259	2.510	2.309	2.149
	8	1.325	1.311	1.299	2.763	2.538	2.357
W36x260	6	1.245	1.232	1.221	2.299	2.121	1.979
	7	1.289	1.276	1.264	2.621	2.409	2.239
	8	1.330	1.317	1.304	2.887	2.65	2.459
W36x245	6	1.247	1.235	1.224	2.376	2.189	2.041
	7	1.292	1.279	1.267	2.712	2.491	2.314
	8	1.333	1.320	1.308	2.989	2.742	2.544
W36x230	6	1.253	1.240	1.229	2.467	2.271	2.115
	7	1.297	1.285	1.273	2.819	2.588	2.402
	8	1.339	1.326	1.314	3.107	2.850	2.643
W36x194	6	1.316	1.302	1.290	2.867	2.628	2.438
	7	1.367	1.353	1.340	3.295	3.016	2.792
	8	1.414	1.399	1.386	3.639	3.333	3.085
W36x182	6	1.324	1.310	1.298	2.990	2.740	2.539
	7	1.374	1.360	1.348	3.436	3.145	2.911
	8	1.421	1.407	1.394	3.792	3.474	3.216
W36x170	6	1.329	1.316	1.303	3.123	2.860	2.649
	7	1.379	1.366	1.354	3.589	3.284	3.039
	8	1.426	1.412	1.400	3.956	3.626	3.356

Table 3 (cont'd.)

Section Type	Slab Thk.	5000	4000	3000	5000	4000	3000
W36x160	6	1.337	1.324	1.312	3.253	2.977	2.755
	7	1.388	1.375	1.363	3.737	3.420	3.163
	8	1.436	1.422	1.410	4.116	3.774	3.494
W36x150	6	1.347	1.334	1.322	3.399	3.109	2.876
	7	1.398	1.385	1.373	3.904	3.573	3.304
	8	1.447	1.434	1.421	4.296	3.940	3.648
W36x135	6	1.376	1.363	1.351	3.690	3.372	3.116
	7	1.430	1.416	1.404	4.236	3.877	3.584
	8	1.481	1.467	1.454	4.653	4.270	3.955
W33x240	6	1.259	1.246	1.234	2.377	2.189	2.040
	7	1.306	1.293	1.281	2.711	2.490	2.313
	8	1.351	1.337	1.325	2.986	2.740	2.542
W33x220	6	1.266	1.253	1.242	2.501	2.300	2.141
	7	1.314	1.301	1.289	2.856	2.621	2.433
	8	1.359	1.345	1.333	3.145	2.886	2.676
W33x200	6	1.277	1.265	1.254	2.658	2.442	2.270
	7	1.326	1.313	1.301	3.038	2.786	2.584
	8	1.371	1.358	1.346	3.343	3.067	2.844
W33x152	6	1.342	1.329	1.318	3.291	3.012	2.789
	7	1.395	1.382	1.370	3.772	3.455	3.197
	8	1.446	1.432	1.420	4.146	3.805	3.526
W33x141	6	1.353	1.340	1.328	3.460	3.166	2.929
	7	1.407	1.394	1.382	3.963	3.631	3.360
	8	1.458	1.445	1.432	4.350	3.995	3.703
W33x130	6	1.368	1.355	1.344	3.670	3.357	3.104
	7	1.423	1.410	1.398	4.200	3.848	3.561
	8	1.477	1.463	1.451	4.603	4.229	3.922
W33x118	6	1.396	1.383	1.371	3.956	3.617	3.343
	7	1.453	1.440	1.428	4.521	4.144	3.836
	8	1.509	1.495	1.482	4.944	4.548	4.219
W30x210	6	1.282	1.269	1.257	2.529	2.326	2.163
	7	1.334	1.320	1.308	2.886	2.649	2.458
	8	1.383	1.369	1.356	3.175	2.915	2.703
W30x190	6	1.290	1.277	1.266	2.685	2.467	2.292
	7	1.342	1.329	1.317	3.066	2.813	2.609
	8	1.392	1.378	1.366	3.369	3.094	2.870

Table 3 (cont'd.)

Section Type	Slab Thk.	5000	4000	3000	5000	4000	3000
W30x172	6	1.295	1.283	1.272	2.848	2.614	2.426
	7	1.348	1.335	1.324	3.251	2.982	2.765
	8	1.399	1.385	1.373	3.568	3.278	3.041
W30x132	6	1.379	1.366	1.354	3.558	3.257	3.014
	7	1.439	1.425	1.413	4.066	3.728	3.452
	8	1.498	1.483	1.469	4.453	4.095	3.799
W30x124	6	1.387	1.374	1.362	3.709	3.395	3.141
	7	1.447	1.433	1.421	4.232	3.883	3.596
	8	1.506	1.491	1.478	4.628	4.259	3.953
W30x116	6	1.397	1.384	1.372	3.877	3.549	3.282
	7	1.459	1.445	1.433	4.419	4.056	3.757
	8	1.520	1.505	1.491	4.824	4.443	4.127
W30x108	6	1.417	1.404	1.392	4.097	3.756	3.468
	7	1.480	1.467	1.454	4.662	4.281	3.968
	8	1.544	1.529	1.515	5.081	4.684	4.353
W30x99	6	1.436	1.423	1.411	4.366	3.998	3.698
	7	1.502	1.488	1.475	4.957	4.556	4.225
	8	1.568	1.552	1.538	5.388	4.973	4.627
W27x177	6	1.315	1.302	1.290	2.761	2.535	2.353
	7	1.374	1.360	1.347	3.150	2.891	2.681
	8	1.431	1.416	1.402	3.459	3.179	2.949
W27x160	6	1.320	1.307	1.296	2.930	2.688	2.494
	7	1.379	1.365	1.353	3.340	3.066	2.843
	8	1.437	1.422	1.409	3.661	3.367	3.125
W27x145	6	1.327	1.314	1.303	3.108	2.851	2.644
	7	1.386	1.372	1.361	3.538	3.249	3.013
	8	1.445	1.430	1.417	3.871	3.563	3.309
W27x114	6	1.403	1.390	1.378	3.800	3.482	3.223
	7	1.469	1.455	1.443	4.320	3.970	3.681
	8	1.537	1.521	1.507	4.710	4.344	4.038
W27x102	6	1.412	1.399	1.387	4.072	3.733	3.456
	7	1.479	1.465	1.453	4.613	4.245	3.940
	8	1.548	1.532	1.518	5.014	4.631	4.311

Table 3 (cont'd.)

Section Type	Slab Thk.	5000	4000	3000	5000	4000	3000
W27x94	6	1.429	1.416	1.404	4.306	3.949	3.657
	7	1.498	1.484	1.472	4.867	4.483	4.163
	8	1.570	1.554	1.539	5.280	4.880	4.547
W27x84	6	1.457	1.445	1.433	4.665	4.282	3.967
	7	1.531	1.516	1.503	5.254	4.846	4.506
	8	1.607	1.590	1.575	5.692	5.264	4.909

4.3 Strengthening of Existing Bridges

Four simple spans were examined to determine the degree to which actual, existing non-composite bridges could be strengthened by making them composite. The example spans were the 40-ft. and 58-ft. spans of the Briarcliff Road bridge in DeKalb County, and the 29-ft. and the 44-ft. spans of the SR-133 bridge over the Seaboard Coast Line Railroad in Dougherty County. The computer program "The Analysis and Design of Simply-Supported Beams for Highway Bridges" developed by the Bridge Division, Georgia Department of Transportation, was used together with manual calculations for determining maximum bending and shear stresses produced by AASHTO truck loadings (1). Table 4 presents the results of the analyses.

The spans were originally designed for H-15 loads with maximum allowable bending stress in the steel beams of 18,000 psi. The bridges have been analyzed for the H-15 load and for both HS-15 and HS-20 assuming non-composite and composite conditions. The most important results are the maximum bending stress in the steel produced from maximum moment conditions and the maximum shear-bond stress at the interface between the steel flange and the concrete deck produced by maximum shear loading conditions. For each analysis all dead load was assumed carried by non-composite behavior of the steel beams; therefore, stresses are the total for dead and live load.

4.3.1 Bending Stresses

For the 29-ft. through 58-ft. spans, the HS-20 loads produced bending stresses greater than the 18,000 psi allowable for the non-composite sections, but the bending stresses were less than allowable for composite sections. In each case making the bridge composite significantly increased its bending load capacity. If the four bridges behaved as composite structures rather than as non-composite, the maximum bending stress under the H-15 load would be reduced 17 percent.

Table 4

Effect of making non-composite bridges composite

Condition	Load	D.L.M.	L.L.M.	L.L.V.	max bending stress in steel σ_{max} (psi)	max shear stress at joint τ_{max} (psi)
Briarcliff Road Bridge, DeKalb County, 40' span, 8" deck, S = 6'4", (W30x108)						
NC	H-15	168	193.2	25.2	14458	0
NC	HS-15	168	252.6	32.9	16835	0
NC	HS-20	165	336.7	46.9	20078	0
C	H-15	168	193.2	25.2	11888	77
C	HS-15	168	252.6	32.9	13482	101
C	HS-20	165	336.7	46.9	15606	138
Briarcliff Road Bridge, DeKalb County, 58' span, 8" deck, S = 6'4", (W36x150)						
NC	H-15	375.2	291.6	27.8	15729	0
NC	HS-15	375.2	424.0	37.7	19032	0
NC	HS-20	375.2	560.3	49.8	22309	0
C	H-15	375.2	291.6	27.8	13880	60
C	HS-15	375.2	424.0	37.7	16128	82
C	HS-20	375.2	560.3	49.8	18440	108
SR-133 Bridge over Seaboard Coast Line Railroad, Dougherty County, 29' span, 7' deck S = 5'3", (W21x68)						
NC	H-15	55.2	110.1	22.0	14155	0
NC	HS-15	53.5	146.8	28.0	17158	0
NC	HS-20	53.5	165.1	37.2	18724	0
C	H-15	55.2	110.1	22.0	10834	117
C	HS-15	53.5	146.8	28.0	12719	150
C	HS-20	53.5	165.1	37.2	13733	199
SR-133 Bridge over Seaboard Coast Line Railroad, Dougherty County, 44' span, 7' deck S = 5'3", (W27x102)						
NC	H-15	153.9	178.0	22.8	14934	0
NC	HS-15	151.3	242.3	31.1	17709	0
NC	HS-20	151.3	322.0	41.9	21291	0
C	H-15	153.9	178.0	22.8	12553	77
C	HS-15	151.3	242.3	31.1	14479	105
C	HS-20	151.3	322.0	41.9	17000	142

The maximum shear stresses at the girder-to-slab interface ranged from 60 psi to 117 psi for the H-15 load and from 108 psi to 199 psi for the HS-20 load. A comparison of these values to the failure shear stress values determined by the push-off tests and by the finite element analysis illustrate the composite action potential of these bridges.

In an actual bridge structure, the shear-bond stress distribution will be different from that for a push-off test. It is believed that the push-off test produces greater stresses at the loaded edge than would be found in at the girder-slab interface of a bridge. The finite element analysis gave an average loaded edge failure stress of 476 psi, while the push-off tests gave a nominal cement-to-steel bond stress of 142 psi and gave a net average steel-epoxy-concrete stress of 202 psi. Because the high leading edge stress probably would not be present in the actual bridge, the actual bond failure stresses would be in the region between the 142 or 202 psi and the 550 psi values.

These bond stress values show that for the four bridge spans under the H-15 loading the spans would respond as composite structures rather than as non-composite structures as designed. The cement-to-steel failure bond stress is greater than the 60 psi to 117 psi shear stress caused by the H-15 load.

The HS-20 loading produces shear stresses in the range of the cement-to-steel failure stress; so that the spans may or may not respond in a composite manner. Possibly near the center of the spans where the shear stresses are lower, the section would behave in a composite manner while near the ends the cement-to-steel bond would fail and the section would be non-composite. If a full width epoxy bond could be achieved, the shear stress produced by the HS-20 load would be less than the average net steel-epoxy-concrete failure stress, and the bridge would respond in a composite manner.

The analysis indicates that the natural bond between the concrete deck and the steel girders produces composite action in these "non-composite" bridges. Under the H-15 design load the safety factor against natural bond failure ranges from about 1.2 to 2.4 based upon the nominal cement-to-steel stress. Under the HS-20 load the shear stresses appear to be at the natural bond failure level. With 100 percent epoxy bonding, the HS-20 induced stresses would be less than the epoxy bond failure level.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The feasibility of using epoxy injection to strengthen existing non-composite bridges is marginal. The Phase I research clearly showed that on an existing bridge the concrete deck could easily be raised and shimmed to create a separation between the deck and the girder, that epoxy could be injected into this gap, and that the epoxy would cure and bond to the steel and concrete. These very positive results demonstrated that epoxy could be used successfully under field conditions during severe winter conditions.

Epoxy could be injected satisfactorily only when the deck was shimmed above the girder; insufficient separation existed for the two cases where the deck was not raised and where the deck was raised and lowered.

Where the deck was shimmed, the epoxy covered a maximum of 70 percent of the steel flange surface. When the deck was raised, pieces of the concrete adhered to the flange. Blowing high pressure air into the gap was insufficient to clean the flange, and the separation was too small for other cleaning methods. Furthermore, the area of the steel flange to which the epoxy bonded was not clean; so the epoxy apparently developed a bond stress capacity less than that demonstrated by previous research discussed in Appendix A. When the epoxy was chipped from the steel, the authors observed that the epoxy was bonded to a thin layer of mill scale which was atop the flange. Therefore, the shear bond capacity was dependent on the adherence of the mill scale rather than on the epoxy. Previous research has shown that the scale must be removed to develop the full strength of the epoxy.

The full potential of the epoxy injection could not be developed because of the mill scale on the steel and because of the adherence of pieces of concrete to the flange. Because the full epoxy bond could not be achieved, development of composite behavior by this method apppears to show only limited success.

An important finding of the Phase I research was that the natural cement-to-steel adhesion is significant and that this bond may be sufficient to develop composite action under working loads. For non-composite bridges, composite action may increase the section modulus of the structure and its load capacity between 10 and 60 percent depending on geometry. More important, such naturally-occurring composite behavior may increase the fatigue life of existing structures because the bending stresses for the composite structures are significantly less than the stresses calculated based upon non-composite action. For four existing bridges designed for H-15 loading, the average difference between composite and non-composite bending stresses was 17 percent.

5.1 Recommendations

Further research into the strengthening of existing bridges using epoxy injection should be delayed because of the marginal feasibility of that technique.

Creation of a clean bonding surface for an epoxy adhesive was demonstrated to be an important condition. A new technique for developing a composite connection between the deck and stringer was conceived during Phase I; this technique assures clean surfaces. A schematic diagram of the connection system is shown in Figure 41. A steel lap plate is epoxy bonded to the underside of the top flange and to the coping of the concrete slab. Prior to bonding, the

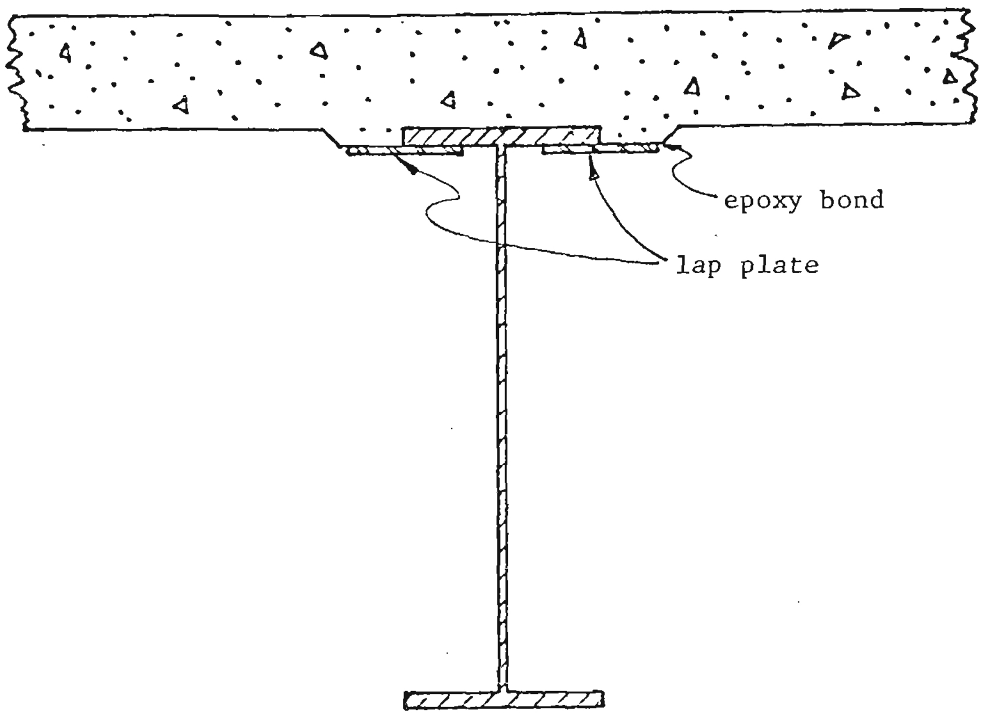


Figure 41. Epoxy bonded steel lap plate for composite connection.

steel and the concrete are sand-blasted and washed to expose clean, solid material and to assure its bondability.

It is recommended that this innovative connection technique be attempted on the Camp Creek Parkway bridge to determine its ease of application.

The existing natural bond between concrete slabs and steel girders deserves further investigation. If it can be shown that existing "non-composite" bridges are behaving as composite structures because of this natural adhesion, the fatigue life of the structure may be lengthened and possibly the allowable load capacity may be increased. Determination of the extent of such bond is difficult. As a first step it is recommended that a service load and an over service load test be conducted on a simple span, non-composite bridge. These tests would resemble Phase II of the current research project. Strain gage measurements would indicate whether the structure was responding in a composite manner. If the natural bond was creating a composite condition, an overload test would be used to fail the bond so that the safety factor of the natural bond could be determined. A theoretical analysis would be compared to the field investigation.

REFERENCES

1. American Association of State Highway and Transportation Officials, Bridge Design Specifications, AASHTO, 1973.

2. Avent, R. R.; Emkin, L. Z.; Howard, R. H.,; and Chapman, C. L.; "Epoxy-Repaired Bolted Timber Connections," Journal of the Structural Division, ASCE, v. 102, n. ST4, April 1976, pp 821-838.

3. Celebi, M., and Penzien, J., "Hysteretic Behavior of Epoxy-Repaired Reinforced Concrete Beams," Earthquake Engineering Research Center, University of California, Berkeley, Report No. EERC-73-5, February 1973.

4. Chung, H. W., "Epoxy-Repaired Reinforced Concrete Beams," Journal of the American Concrete Institute, v. 72, n. 5, May 1975, pp 233-234.

5. "FHWA reports to Congress on National Bridge Problem," Better Roads v. 47, n. 5, May 1977, pp. 19-22.

6. Hearings, Subcommittee on Surface Transportation, House Committee on Public Works and Transportation, Ninety-fourth Congress, (94-74), September, 1976.

7. Herr, L. A., Speech at the 23rd Highway Engineering Conference, Los Cruces, New Mexico, April 14, 1977.

8. Kahn, L. F.; Townsend, W. H.,; and Kaldjian, M. J.; "Ferrocement Steel Plate Composite Beams," Journal of the American Concrete Institute, v. 72, n. 3, March 1975, p. 94-97.

9. Kajfasz, S., "Concrete Beams with External Reinforcement Bonded by Glueing, Preliminary Investigation," RILEM International Symposium, Paris, September 4-6, 1967.

10. Kratzer, J. T., letter to H. L. Tyner, dated October 17, 1977.

11. Miklofsky, H. A.; Brown, M. R.,Jr.; and Gonsior, M. J.; "Epoxy Bonding Compounds as Shear Connectors in Composite Beams," Physical Research Project No. 13, Engineering Research Series RR62-2, Department of Public Works, State of New York, June 1962.

12. Miklofsky, H. A.; Gonsior, M. J.; and Santini, J. J.; "Further Studies of Epoxy Bonding Compunds," Physical Research Project No. 13, Engineering Research Series RR63-5, Department of Public Works, State of New York, July 1963.

13. "One of Six U. S. Highway Bridges Is Deficient," Engineering News-Record, v. 198, n. 10, March 10, 1977, pp. 18-21.

REFERENCES (continued)

14. Pollock, R., personal communications, 1977.

15. Schutz, R. J., "Epoxy Adhesives in Prestressed and Precast Concrete Bridge Construction," Journal of the American Institute, v. 73, n. 3, March 1976, p. 155-159.

Appendix A. Background

A.1. Need For Bridge Repair And Strengthening

A major issue facing the nation today is the deteriorating condition of our highway bridges, the so-called "bridge crisis" (5,6,7,13). According to the Federal Highway Administration (FHWA), more than one of every six highway bridges is either structurally unsound, has a badly deteriorated deck or is inadequate to handle traffic demand (5). Current data on bridges in the federal aid system and a projection of limited information for bridges off the system shows that there are 9003 structurally deficient* bridges and 30,917 functionally obsolete** units in the federal highway system (5). This is based on a national bridge inventory which is about 98% complete. There are about 65,600 deficient bridges in the off federal-aid system for a total of about 105,500 deficient bridges nationwide (5,7,13).

One obvious solution is total replacement of all bridges. This type of action would cost an estimated $12.4 billion for federal-aid system bridges and another $10.6 billion for off-system bridges. The total cost would amount to $23 billion (5,7,13). Presently the Federal Bridge Replacement Programs is funded at $180 million annually. At this rate, it would take well over 50 years to replace the 30,917 deficient bridges on the Federal-aid system alone (7). Clearly the budget of the Federal Bridge Replacement Program must be drastically increased or another solution must be employed.

* A structurally deficient bridge is one that has been restricted to light vehicles only, or closed.

** A functionally obsolete bridge is identified as one whose deck geometry, clearance or approach roadway alignment is not adequate for the system of which it is an integral part.

Repair and strengthening of existing bridges is such an alternative, one which is economically sound, and structurally feasible.

A.2. Previous Research On Epoxy-Concrete Construction

Using epoxy adhesives for repair and strengthening has been effective for many civil engineering applications and appears applicable for bridge maintenance. Some previous research on using epoxy for repair and strengthening is discussed below (2,3,4,8,9,11,12,15). Much information concerning the effectiveness of epoxy repair has gone unpublished (14).

A limited amount of past research indicates that epoxy bonding may be used to join steel beams and concrete slabs to form a composite structure. Miklofsky, et al. (11,12) tested nine composite beams which were constructed using W10x21 steel sections and 4-inch thick concrete slabs poured-in-place. The difference in construction of the nine beams was the type of shear connection used between the steel beam and the concrete. Three beams used an epoxy which was spread prior to casting the slab; three used standard Nelson stud shear connectors; and three used no mechanical connection device. The beams made without shear connectors showed no composite behavior. The beams made using epoxy bonding showed ultimate strengths ranging from over 90 percent to 100 percent of the strengths of those beams made using studs. The strain response of the epoxied beams shows excellent structural interaction between the beam and slab. Miklofsky, et al. (11,12) concluded that the epoxy provides an excellent shear connection although the epoxy may not be as good as studs under repeated loads.

Kahn, Townsend and Kaldjian (8) tested thin composite slabs which were constructed using epoxy or studs as the shear connector between a steel plate and a concrete section. The epoxy provided sufficient shear resistance to develop full composite action and to permit a tension-yield failure of the slab. The epoxied slabs showed similar ultimate strength as the studded slab, but the former demonstrated less ductility.

Schulz (15) has described the recent construction of a composite highway bridge built on the New York Thruway. Precast concrete deck sections with blockouts for steel shear connectors were epoxy bonded to the top flange of steel stringers. Later, steel plate shear connectors were welded to the stringers, and the blockouts were filled with epoxy mortar. Load tests of this bridge indicated complete composite action.

Kajfasz (9) strengthened existing reinforced concrete beams by epoxy bonding steel reinforcing bars or steel plates to the tension face of those beams. Tests showed that the resulting section demonstrated full composite behavior and that the beams failed by yielding of the steel.

This limited number of investigations on the behavior of composite beams made using epoxy bonding has demonstrated the potential of epoxy as a shear connector. None of these studies investigated connecting an existing slab to its supporting stringer.

Pressure grouting with epoxy has been shown as an excellent means of bonding cracked sections of existing reinforced concrete (3,4), and of timber (2). Pressure grouting epoxy to the interface of an existing concrete slab and steel beam appears to be a promising means of connecting the two materials.

APPENDIX B: Further Details of the Investigation

B.1.0 Jacking System

Experimental investigation required that the slab over two of the three girders had to be raised off of these girders prior to injection. The first step was the design of an apparatus to accomplish the lifting task.

B.1.1 System Requirements

Criterias for the design of the lifting apparatus were the magnitude of the force required to cause the slab to separate from the steel girder and the ability to manually operate the apparatus. A number of contributing factors had to be evaluated to estimate the lifting force; these included the dead load of the concrete slab, natural adhesion between concrete and steel, and the friction effect of the coping. These factors could not be well defined, for example the amount of concrete which would arch off the flange was not known because of the difficulty in calculating this nonlinear phenomenon.

The initial selection for the apparatus was to use two 20 ton jacks (Enepac Model RC-251). The selected bridge had very stiff steel stringers with the thickness of the bottom flanges measuring up to three inches. In addition to the flanges and web, the girder had stiffness at 5 ft. intervals and had several diaphrams made from steel angle sections. It was therefore, feasible, considering the capacity of the system, to jack against the bottom flange of the stringers in order to raise the slab. Jacking form this position would cause the slab to be pushed up while the stringer would be pushed down.

B.1.2 Design requirements

Positioning the jacking system to facilitate this type of action required that the base of the jacking system span the distance between the webs of the steel stringers (6 ft. 6 in.), and that the apparatus be sufficiently light

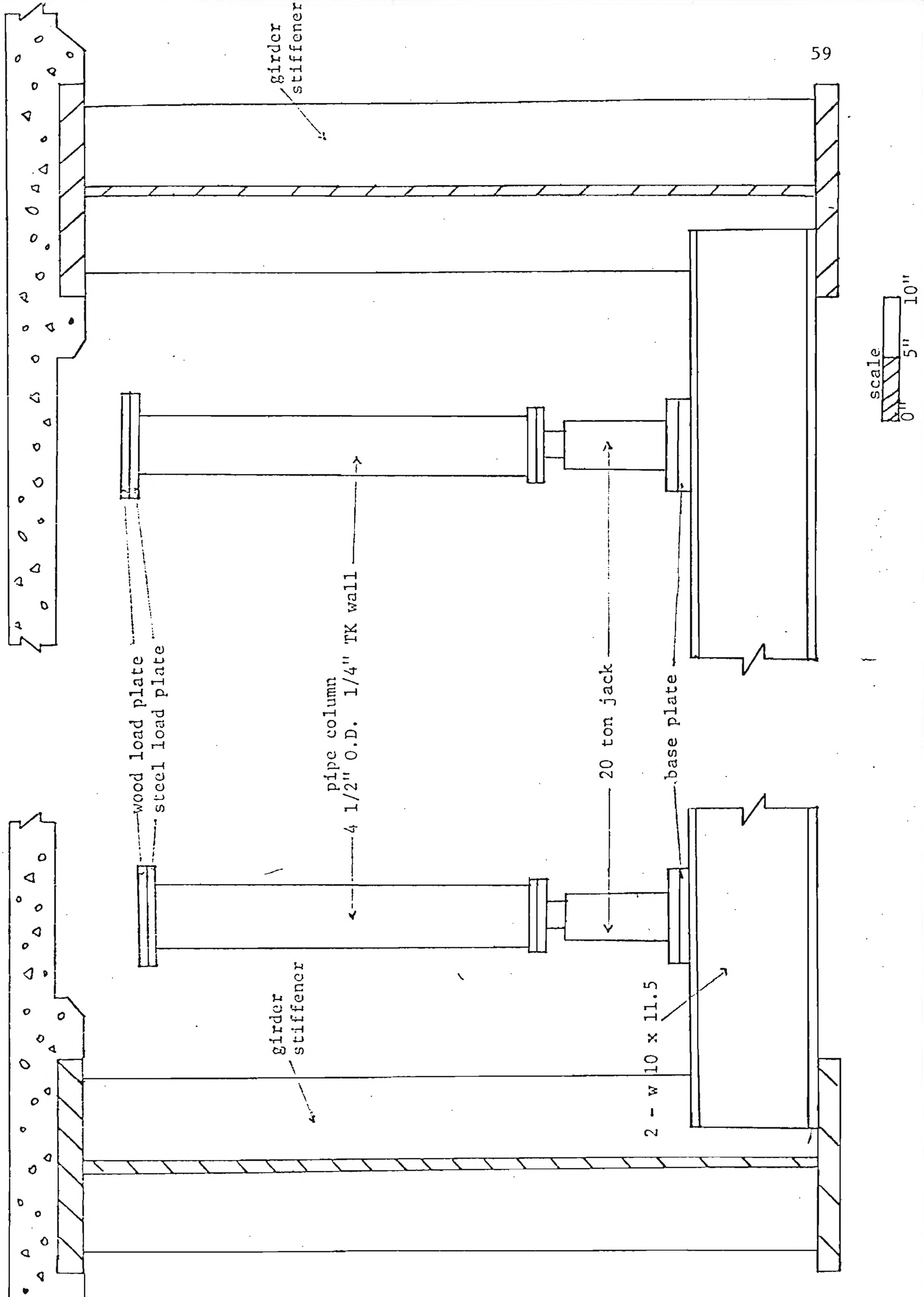


Figure B.1: Initial Jacking System

to be assembled and moved by two men.

The estimated total weight of the jacking system was large enough to make field assembly necessary. Variable placement of the system in the field was also required because the length of the girder to be injected was 10 ft. For these reasons the jacking system had to be designed so that it could be easily assembled and disassembled in the field. The weights of the heaviest components were kept less than 120 lbs. so that they could be handled easily by two men.

B.1.3 Fabrication of Jacking System

The jacking system was fabricated using various sized steel plates, thick walled 6 in. diameter structural pipe, and two w10 x 11.5 wide flange sections. Figure B.1 shows this arrangement. The entire system was shop assembled and tested, and then disassembled prior to shipment to the field.

Calibration of the jacking force was achieved by measuring in a laboratory the applied force from the jack using a universal testing machine and by reading the corresponding pressure registered on the hydraulic gage attached to the jack.

B.1.4 Field Jacking

The jacking system was assembled so that the loading plates were approximately three inches from the edge of the coping. Jacks were loaded with the same approximate force to insure an even vertical displacement in the slab. Upon reaching the safe capacity of each jack, which was 30 kips, no visual signs of separation could be noted. The natural adhesion and friction between the coping and the edge of the flange apparently had been underestimated. A slightly higher load caused on pipe to start to bend laterally. The loads were removed. A decision for a higher capacity system to be designed was then made as well as an improved design to prevent lateral bending of the pipes.

B.1.5 Development of Jacking Apparatus

The requirement of capacity increase was satisfied by replacing one of the 20 ton jacks by a 50 ton jack (Blackhawk Model R210) as shown in Figure B.2. Due to the column bending problem experienced in the first testing, the new design employed a ball-joint concentric loading plate system. This design option would insure that a concentric load would be applied to the pipe column. The increase in jack capacity caused the need for larger support beams; two W 12 x 19 were chosen. The new system was calibrated in the same manner as the first.

Placement of the jacks was changed. The first method loaded the slab at points next to different girders, however the new method positioned the jacks on opposite sides of the same girder; this concentrated the lift force into a smaller zone as shown in Figure B.3.

B.2.0 EPOXY PROCEDURES

Initial sealer application occurred on November 26, when weather conditions were less than favorable due to low temperatures (average 35°F) and high moisture content of the air. The sealer was prepared by mixing the two components with a electric drill with stir-rod attachment (Figure B.4). The mixed sealer was placed in plastic tubes, like the one in Figure B.5, and then raised to the top of the scaffolding. Putty knives as shown in Figure 9 were used in the application; due to the chemical nature of the sealer, rubber surgical gloves were constantly worn by all personnel.

One problem encountered was the excessive moisture which collected on the steel girders; this condition made adhesion between the steel and sealer very poor. To remedy the situation the steel was dried using a towel and electric 1000 watt dryer (Figure B.6).

Highly potential leakage areas such as around ports and honey combed concrete

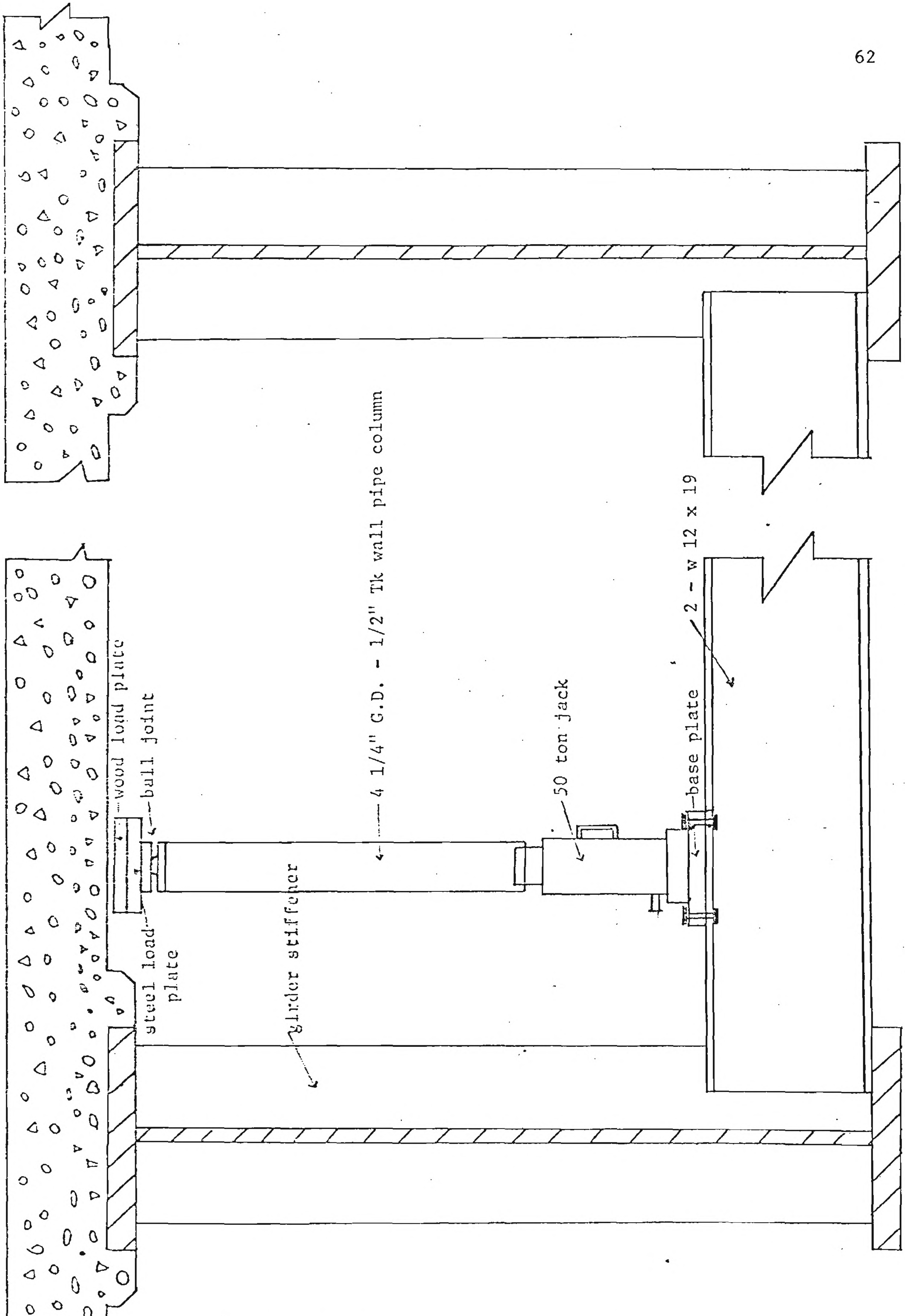


Figure B.2: 50 Ton Jacking System

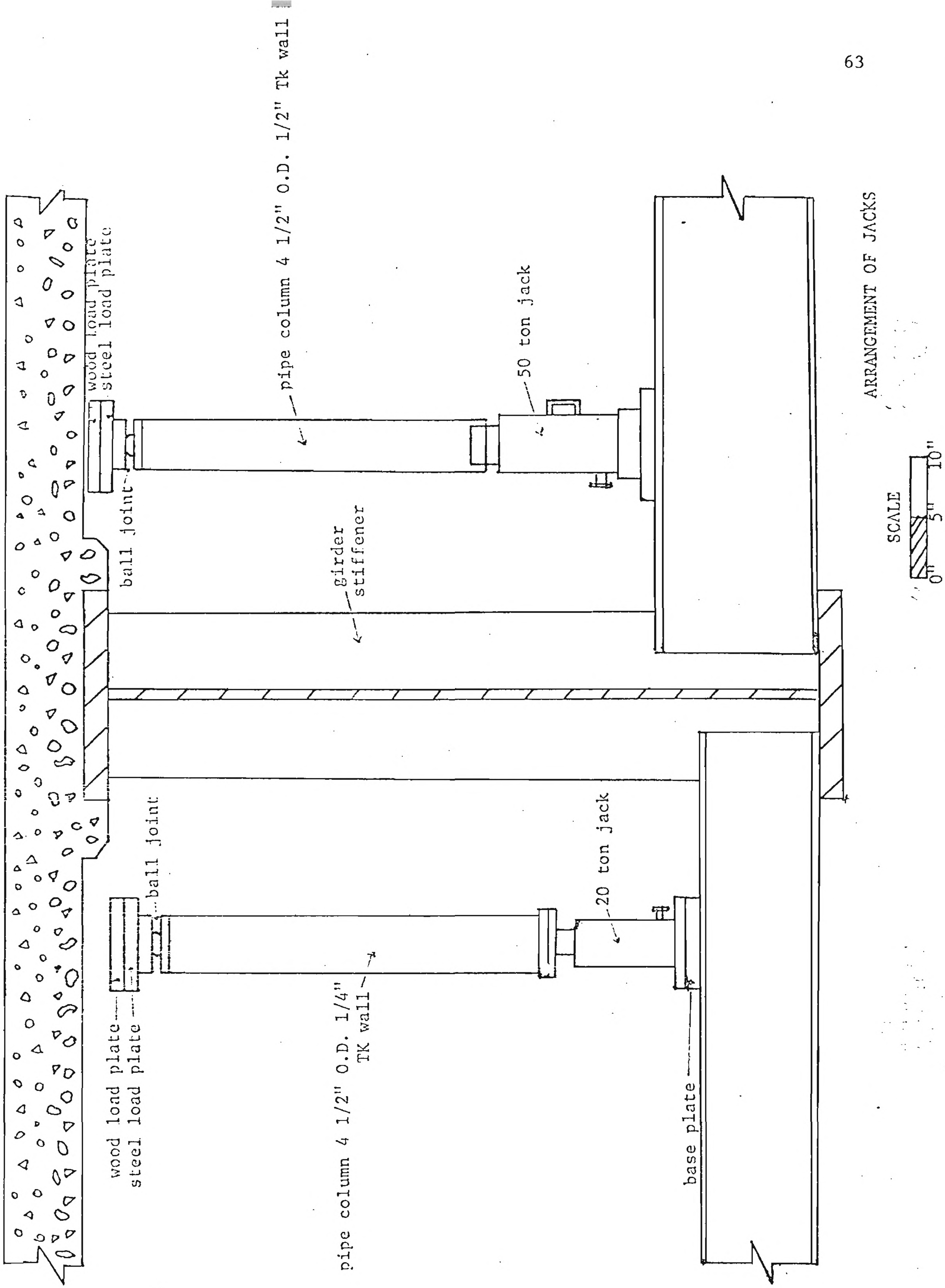


Figure B.3: Modified Jacking System

Figure B.4: Mixing epoxy sealer

Figure B.5: Preparation for Application of Sealer

Figure B.6: Drying procedure used on Girders sections were given special attention in the sealing operation. The sealer was applied all along the girder-to-slab joint for the entire testing length and beyond.

B.3.0 DECK SAWING

The slab above each girder was cut into suitably sized sections for the purpose of bond testing. Layout of the three girders outlining the actual section sizes are pictured in Figures B.7, B.8, and B.9 .

B.4.0 ANALYTICAL INVESTIGATION

Stresses along the concrete-steel interface caused by the push-off force would seem to vary in magnitude from the load face to the backside face. The extent of the variation is important when considering the shear stress capacity of composite action in an actual bridge. A finite element analysis was made using the Georgia Institute of Technology Integrated Civil Engineering System STRUDL II program.

The modeling grid used to represent the concrete section is pictured in Figure B.10. This grid is one plane of the 13 planes used to construct the 3-dimensional model. Plane layout is outlined in Figure B.11. The smaller grid along the concrete steel interface was used to produce more accurate results. The IPLSCH element, an eight node, 3-dimensional rectangular elastic element, was used in the analysis.

Support conditions of complete translation restraint were set at all nodes on the steel-slab interface plane. Elastic material properties were chosen that closely compared with actual concrete properties of the tested bridge slab.

The failure load of Girder 1 Section 6 was 55,000 lbs. This number was read off the calibrated pressure gauges during the push-off tests. This same load was divided down into nodal point loads that could be used in the finite element analysis. Nodal load positions and magnitudes were calculated by superimposing

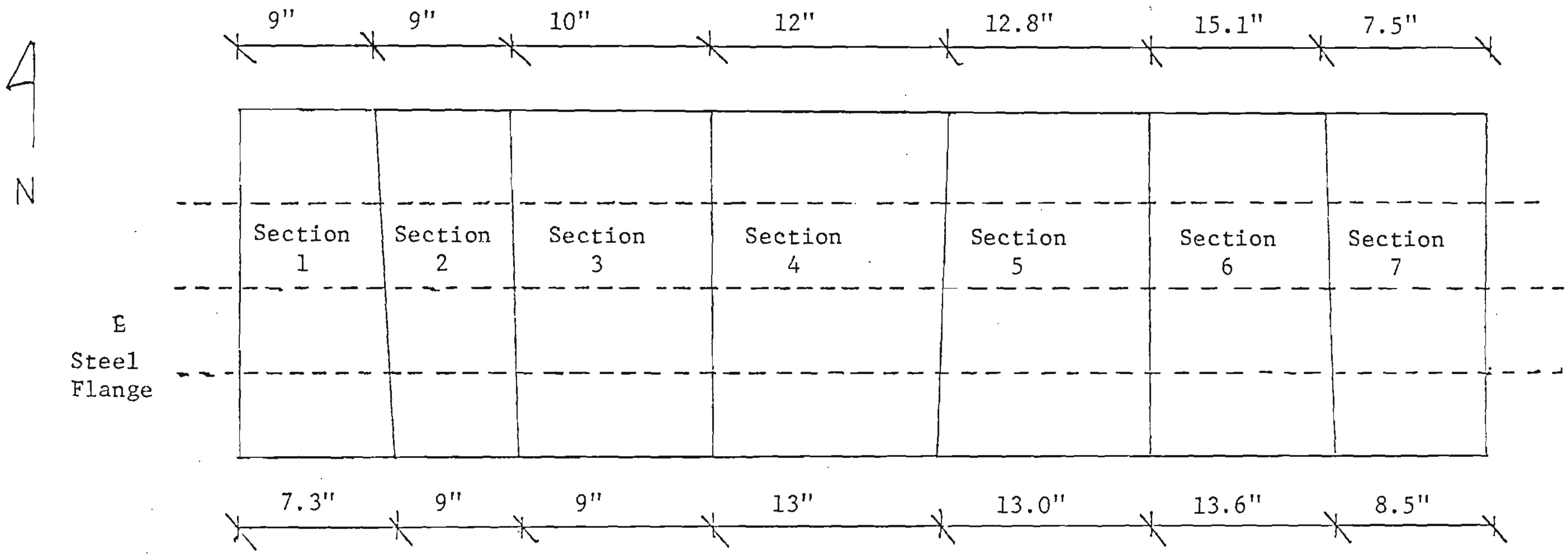


Figure B-2: Girder 2-Section Layout

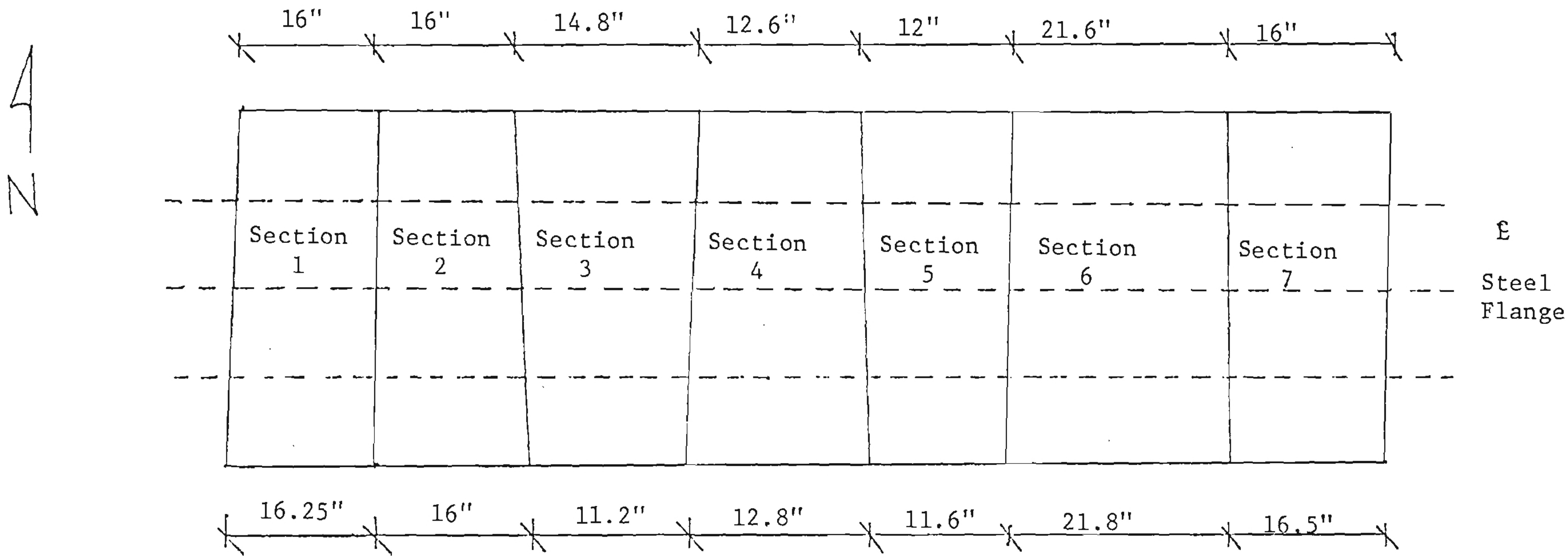


Figure B8 : Girder 1-Section Layout

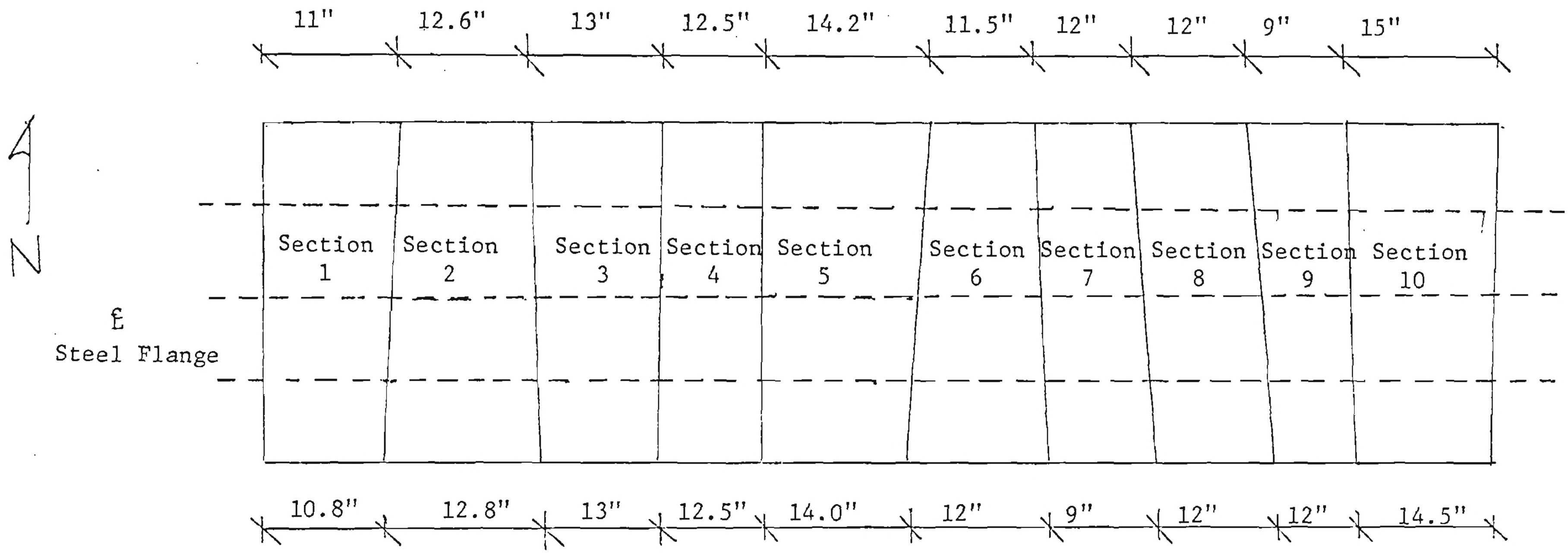


Figure B-9: Girder 3-Section Layout

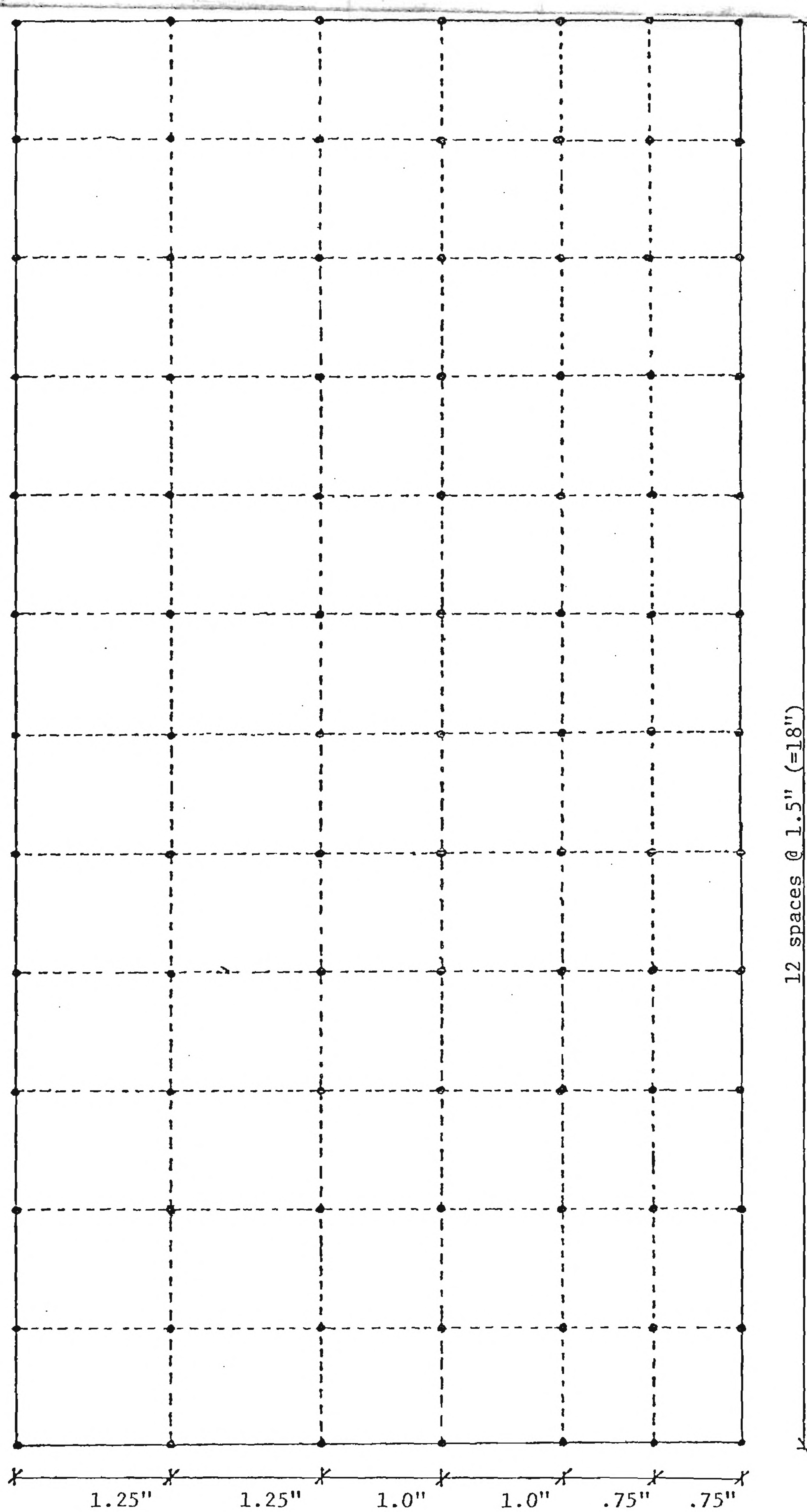


Figure B.10: Finite Element Grid

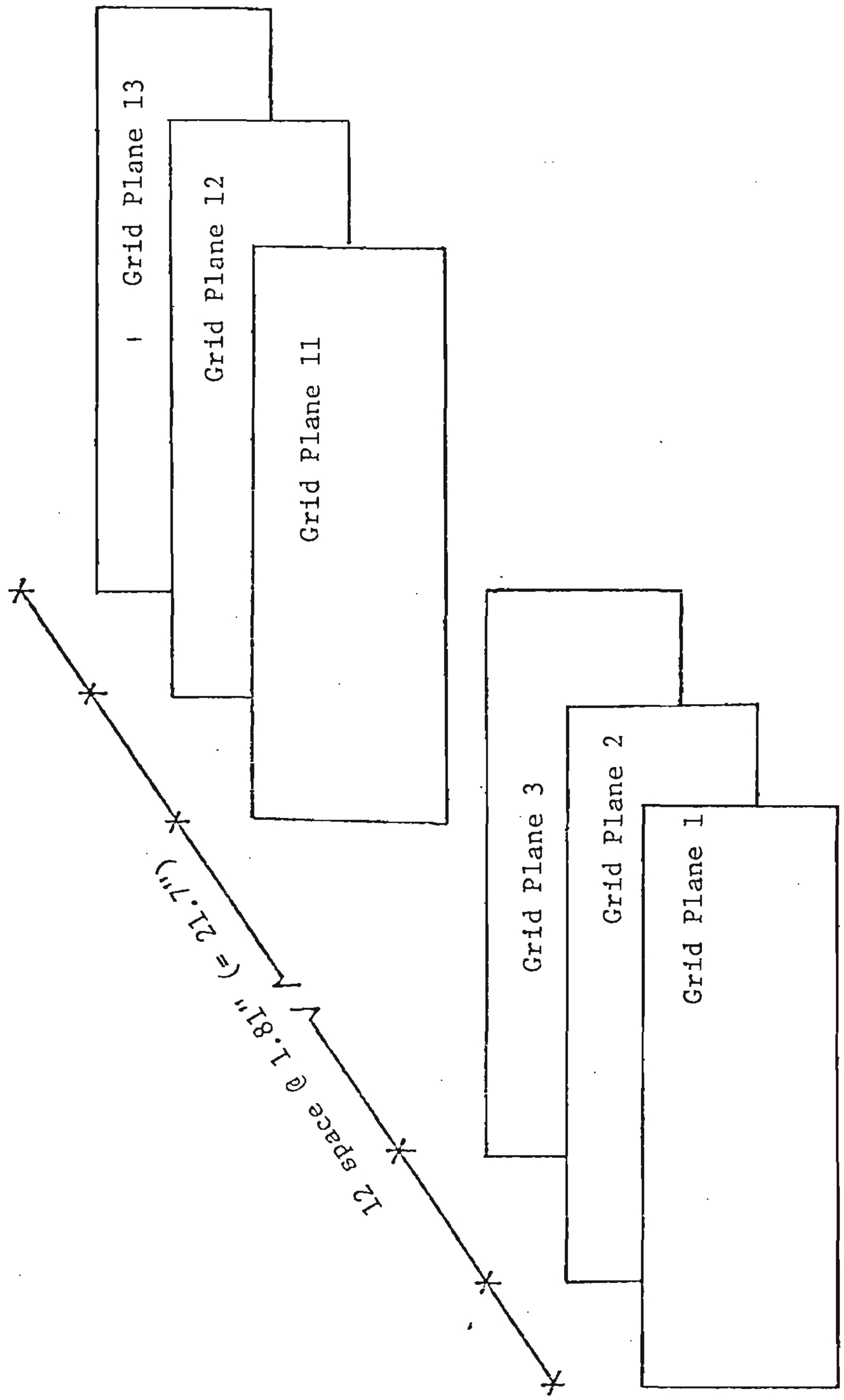


Figure B.11: Grid Plane Layout for Finite Element Model a load plate diagram, which was the same size as used in field, over the finite element grid.

Results of the analysis are given in Section 4.1 of the report.

Appendix C. Climatological Data

TABLE C-1. CLIMATOLOGICAL DATA (1)

MONTH	DAY	MAXIMUM TEMP (°F)	MINIMUM TEMP (°F)	AVERAGE TEMP (°F)
November	(2) 26	43	26	35
	27	57	26	42
	28	51	41	46
	29	57	48	53
	30	56	53	55
December	1	64	42	53
	2	55	38	47
	3	65	38	52
	4	70	55	63
	5	75	47	61
	6	47	26	37
	7	34	15	25
	(3) 8	49	24	37
	(4) 9	49	22	36
	10	38	16	27
	11	48	19	34
	12	50	25	38
	13	58	41	50
	14	68	48	58
	15	65	43	54
	16	52	40	46
	17	54	49	52
	18	57	38	48
	19	64	34	49
	20	65	36	51
	21	41	24	33
	22	43	25	34
	23	50	28	39
	24	57	45	51
	25	53	28	41
	26	34	19	27
	27	45	17	31
	28	39	17	28
	29	47	21	34
	30	36	34	35
	31	49	36	43
January	1	49	35	42
	2	37	25	31
	3	44	20	32
	4	53	22	38

(1) Data provided by National Weather Service or Hartsfield International Airport.
(2) Date of sealer application
(3) & (4) Dates of injection.

MONTH	DAY	MAXIMUM TEMP (°F)	MINIMUM TEMP (°F)	AVERAGE TEMP (°F)
January	5	55	30	43
	6	54	42	48
	7	57	50	54
	8	60	29	45
	9	29	10	20
	10	25	11	18
	11	35	9	22
	12	33	21	27
	13	31	28	30
	14	29	17	23
	15	31	18	25
	16	44	18	31
	17	46	35	41
	18	43	31	37
	19	43	35	39
	20	35	29	32
	21	36	26	31
	22	46	27	37
	23	53	26	40
	24	45	34	40
	25	61	35	48
	26	38	20	29
	27	29	19	29
	28	38	18	28
	29	35	17	26
	30	41	18	30
	31	40	25	33